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PROCEEDINGS, PART II

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COHESION AND ATOMIC STRUCTURE

EDGAR MARBURG LECTURE

BY SAUL DUSHMAN[1]

SYNOPSIS

There exists a relationship between at least four cohesional properties of the elements and electronic configuration of the atoms. The crystal structure of pure metals also shows a similar connection with electronic arrangement and cohesional forces.

While a theory of cohesion in solids based upon atomic structure has not yet been formulated, it is possible to calculate the magnitude of cohesive forces by introducing plausible assumptions regarding the laws of attraction and repulsion between atoms. As shown by Grüneisen for the case of pure metals and by Born for ionic lattices (such as rock salt) there must exist definite relations between the theoretical cohesive force between atoms and such experimentally observed values as density, latent heat of evaporation and coefficients of expansion and compressibility. The cohesive forces calculated on the basis of this theory are found to be from ten to one hundred times greater than the rupture strengths actually observed.

In explanation of this discrepancy, Griffiths and Smekal suggested the presence of fine cracks, crevices, or inhomogeneities in the crystal structure. Various observations made by these and other investigators are found to be in good accord with such an assumption. The theory has been extended quite recently by Zwicky and evidence adduced to show that the distribution of such cracks must be governed by the lattice structure and that this accounts for the fairly definite values observed technically for rupture strength of different materials.

The effect of temperature on the rupture strength is discussed at length. A plot of the observed data against temperature shows two distinct portions, one below a definite inflection temperature, which is approximately linear, and the other, which extends from the inflection temperature upwards and shows a very

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much more rapid decrease in strength with temperature. In explanation of these observations an extension is suggested of the theory previously postulated by Jeffries and Archer with regard to the relative strength of crystalline and intercrystalline material. Above a certain critical point deformation occurs by viscous flow, and consequently the rate of decrease in tensile strength with temperature is dependent upon the rate of decrease of the coefficient of viscosity with increase in temperature.

Analogy with other reactions, both chemical and physical, whose rates change with temperature in the same manner, suggests that from the temperature coefficient of the rupture strength, it should be possible to calculate the energy required to cause relative translational motion of atoms or grains such as occurs in plastic flow of solids. The values of the energy thus calculated are found to lie between the heats of fusion and evaporation.

Similarly, from the rate of grain growth, rate of diffusion, electrolytic conductivity of salts, viscosity of glass, rate of flow of metals, each of which changes rapidly with temperature, it is possible to calculate the energy involved in the reaction. Thus the study of the effect of temperature on all these phenomena furnishes a means of determining the relative magnitude of cohesive forces involved.

INTRODUCTION

In the introductory chapter of his recent book on "The Nature of the Physical World," A. S. Eddington emphasizes in a very interesting manner that every article about us has a dual aspect from the scientist's point of view. One of these aspects is that which is familiar to us from earliest childhood. A table, or a chair, has something which has extension, is substantial, tangible, and so forth. The other aspect is, as he states, one which is forced upon us as a result of scientific investigation on the nature of matter. The table becomes an aggregate of atomic nuclei between which electrons are rushing about with immense speed, and the actual volume occupied by both electrons and nuclei is so small compared with the physically measured volume of the table or chair that we must regard the latter as being largely empty space.

I should like to apply this idea to the subject under discussion in this paper. Let us consider a bar of pure copper. To the mechanical engineer this bar is merely a piece of metal which has a definite shape, mass and tensile or compressive strength. To the metallurgist it becomes of interest to view the metal under powerful magnification and he sees in the grain structure of the metal certain features which reveal further information regarding the mechanical properties of the bar. To the physicist, however, the piece of copper reveals a microcosm which is astounding in its complexity and replete with some of the most profound problems with which human intellect has ever been confronted. Where the metallurgist under his one thousand fold magnification sees odd-shaped crystals, the physicist, by means of the invisible X-radiation, which permits him to magnify, as it were, to one hundred million fold magnification, perceives an orderly arrangement of atoms constituting a crystal lattice. Furthermore, he finds it possible to delve into the very inside of the atom itself and there is revealed to him an orderly system of a nucleus and its electrons compared to which the so-called harmony of the celestial spheres is only a feeble analogy.

Now it may well be asked, what is the connection between what the physicist concludes as to the microstructure of copper with what the engineer reports as the macroscopic behavior of a copper bar? This is the problem which you have asked me to discuss before you to-day, and while I must tell you at the very beginning that the physicist is as yet unable to give even a moderately satisfying answer, it is possible to "explain," in terms of atomic physics, some of the observed facts about the strength of materials.

I put the word "explain" in quotation marks because, before proceeding further, and in order that misunderstanding may be avoided, I wish to state more definitely what is meant by this word when used scientifically. As was first pointed out by a German physicist, Mach, theories in physics are not "explanations" of observed phenomena in the philosophical or metaphysical sense of the word. That is, these theories contain no implicit conclusions regarding some reality *back* of the phenomena. Rather, the object of a physical theory is to *develop a system of conceptions which will enable us to describe the results of an experience*—a system which shall be as complete and comprehensive as possible and at the same time extremely simple. Or as a famous mathematician has expressed it: The object of science is not to explain phenomena, but to learn the laws which govern these phenomena. Hence the physicist cannot explain why copper has a certain tensile strength under certain conditions, but he can attempt to discover some relations between the tensile strength and the electronic distribution in the copper atom—relations which shall at the same time enable him to derive conclusions regarding other metals which are also in agreement with observed facts.

ATOMIC STRUCTURE AND PERIODIC PROPERTIES OF THE ELEMENTS

As a result of the work of Rutherford, Moseley, Aston and others, the conclusion was deduced in 1912 that the atom consists of a positively charged nucleus of extremely small magnitude compared to the diameter of the atom itself, with a number of electrons or negatively charged corpuscles external to the nucleus, the combined charge on the electrons being equal and opposite to that on the nucleus. The number of electrons or number of units of positive charge on the nucleus corresponds to the position of the element in the periodic table and is defined as the *atomic number*. Thus hydrogen, the element of atomic number $N = 1$, has one electron; helium, $N = 2$, has two electrons, and so forth for the other elements, until we reach uranium, the heaviest atom known, for which $N = 92$.

It is obvious that the various properties of the elements must be intimately dependent upon the number and arrangement of the electrons in the different atoms, and since, as is well known, a large number of these properties show periodic variation with increase in atomic number, the subject of electronic arrangement becomes of extremely great interest.

Our present views on this are based mainly on the epoch-making suggestions postulated by N. Bohr in 1913 and subsequently developed by A. Sommerfeld, M. Born and others.[1, 2] The Bohr theory is essentially an explanation of the origin of spectra in terms of electronic "energy levels." The main postulates of this theory may be stated briefly as follows.

1. An atomic system can exist only in a series of states corresponding to a discontinuous series of values for its energy, and consequently any change of energy of the system, including emission and absorption of electromagnetic radiation, must take place by a complete transition between two such states. These states will be denoted as the "stationary states" of the system.

2. Whenever a transition occurs from one state to another, the energy absorbed or emitted is in the form of monochromatic radiation of frequency ν, given by the quantum theory relation

$$h\nu = E_i - E_f$$

where E_i and E_f designate the energy values corresponding to initial and final states respectively, and h is Planck's constant.

Bohr furthermore postulated a certain rule by which the nature of the discrete stationary states could be determined in the case of any given atomic system.

The essential point in Bohr's theory, and it is one which must remain unaltered no matter what subsequent alterations the theory

---

[1] A summary of the earlier development of the Bohr theory will be found in the chapter by the writer in H. S. Taylor's "Treatise on Physical Chemistry," Vol. 2, pp. 1079–1130, Van Nostrand, New York City, 1924.

[2] The more recent applications of the Bohr theory to spectroscopy and atomic structure are discussed in the paper by S. Dushman, "Line Spectra and the Periodic Arrangement of the Elements," *Chemical Reviews*, Vol. 5, p. 109 (1928).

may undergo, is that the various spectral lines, emitted by an atom, whether these be in the X-ray region, or visible or infra red, are due to transitions in the electronic arrangement from one configuration to another, and the possible number of these configurations is governed by certain laws. We may illustrate this point of view by referring to the simplest atom, that of hydrogen.

According to Bohr, the normal hydrogen atom consists of an electron rotating in a circular orbit around a unit positive charge. Bohr assumes that the electrostatic force of attraction between nucleus and electron is balanced by the centrifugal force on the latter, thus maintaining the system in a state of stable equilibrium. If, however, an electron from an external source, as, for instance, from an incandescent tungsten filament, is made to collide with a hydrogen atom, and if the speed of the impinging electron is sufficiently great, the electron in the atom is knocked out of its normal orbit into an orbit of larger radius, where its potential energy with respect to the nucleus is consequently higher. The hydrogen atom is no longer in the normal state—it is said to be in an "excited state"—and will immediately tend to return to the normal state. In this return the energy stored as a result of the previous collision is re-emitted in the form of monochromatic radiation, that is, as a spectral line.

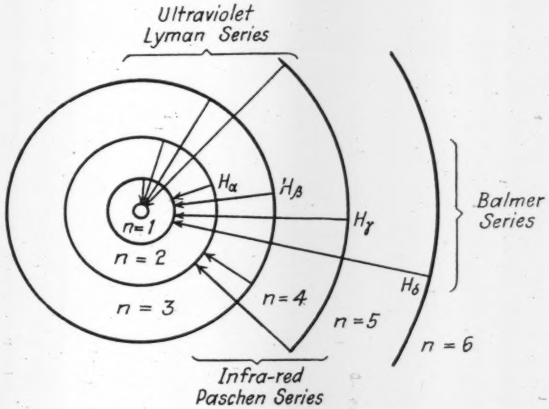


FIG. 1.—Electron Orbits in Hydrogen Atom, Illustrating Origin of Spectral Series Lines as Transitions Between Orbits.

The different possible orbits are designated by *quantum numbers*, n, with integral values ranging from $n = 1$ to $n = \infty$, and are essentially "energy levels." Figure 1 shows different orbits in the hydrogen atom, and also the origin of different spectral series. Only the innermost orbit ($n = 1$) is that which corresponds to the position of the electron in normal hydrogen. All the other orbits external to this are virtual orbits and therefore correspond to possible positions of the electron in excited hydrogen atoms. It should also be mentioned in this connection that Bohr was able to calculate *quantitatively*, on the basis of this theory, the frequencies of the different lines in the spectrum of hydrogen, and to account in a logical manner for certain empirical generalizations which had been deduced previously by spectroscopists.

The simple Bohr theory of the hydrogen atom was subsequently modified by Sommerfeld to include the possibility of elliptic as well as circular orbits and later investigations showed that it is also necessary to postulate that the various orbits may be oriented in different planes with respect to an axis of symmetry in the nucleus.

TABLE I.—DISTRIBUTION OF ELECTRONS IN DIFFERENT TYPES OF LEVELS.

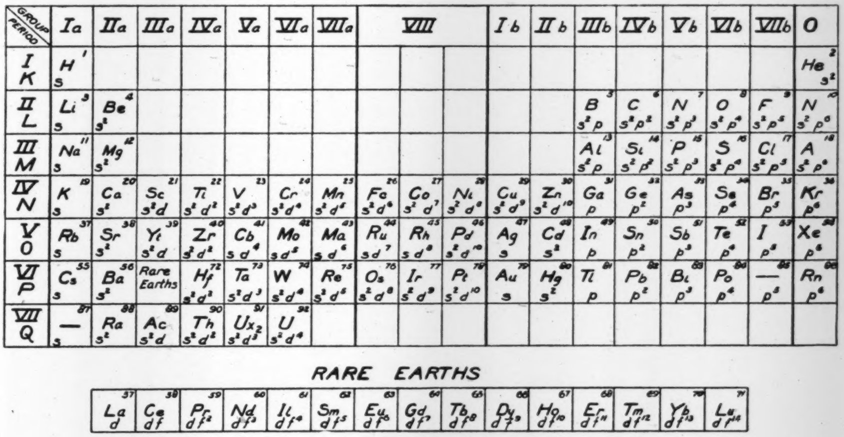
QUANTUM NUMBER, n	QUANTUM NUMBER, l	X-RAY DESIGNATION	OPTICAL DESIGNATION	MAXIMUM NUMBER OF ELECTRONS
1	0	K	1*s*	2
2	0	L_I	2*s*	2
	1	$L_{II\text{-}III}$	2*p*	6
3	0	M_I	3*s*	2
	1	$M_{II\text{-}III}$	3*p*	6
	2	$M_{IV\text{-}V}$	3*d*	10
4	0	N_I	4*s*	2
	1	$N_{II\text{-}III}$	4*p*	6
	2	$N_{IV\text{-}V}$	4*d*	10
	3	$N_{VI\text{-}VII}$	4*f*	14
5	0	O_I	5*s*	2
	1	$O_{II\text{-}III}$	5*p*	6
	2	$O_{IV\text{-}V}$	5*d*	10
	3	$O_{VI\text{-}VII}$	5*f*	14
	4	$O_{VIII\text{-}IX}$	5*g*	18

Now what happens when two or more electrons are present in the atom? The conclusions regarding this question may be stated best in the form shown in Table I.

Corresponding to quantum number $n = 1$, it is possible to have only two electrons in circular orbits. Thus in helium each electron occupies a circular orbit similar to that occupied by the single electron in hydrogen, but owing to increased nuclear charge, $N = 2$, the radii of the orbits will be much smaller; as more electrons are added externally to these two and the nuclear charge is increased, the radii of these innermost electrons become smaller and smaller. In lithium,

$N = 3$, the third electron assumes an orbit of larger radius, in fact the orbit is elliptic and the value of the second quantum number, l, is a measure of the eccentricity—the smaller the value of l the smaller the ratio between minor and major axis. The next element is beryllium, $N = 4$, and the fourth electron assumes an orbit similar to that of the added electron in lithium.

But the maximum number of electrons of type $n = 2$, $l = 0$ is two. Hence with boron, $N = 5$, we begin a new group of orbits ($n = 2$, $l = 1$) and this group is completely filled when we come to neon, $N = 10$. The symbols, K, L, M, correspond to the designation of these energy levels as derived from X-ray spectra. In the building up of atoms by addition of successive electrons, the orbits which are virtual for the elements of lower atomic number gradually become filled up by the added electrons, and the last column in Table I represents the maximum number of electrons of that type which an atomic system can possess.



GROUP / PERIOD	Ia	IIa	IIIa	IVa	Va	VIa	VIIa	VIII			Ib	IIb	IIIb	IVb	Vb	VIb	VIIb	O
I K	H 1 s																	He 2 s^2
II L	Li 3 s	Be 4 s^2											B 5 s^2p	C 6 s^2p^2	N 7 s^2p^3	O 8 s^2p^4	F 9 s^2p^5	N 10 s^2p^6
III M	Na 11 s	Mg 12 s^2											Al 13 s^2p	Si 14 s^2p^2	P 15 s^2p^3	S 16 s^2p^4	Cl 17 s^2p^5	A 18 s^2p^6
IV N	K 19 s	Ca 20 s^2	Sc 21 s^2d	Ti 22 s^2d^2	V 23 s^2d^3	Cr 24 s^2d^4	Mn 25 s^2d^5	Fe 26 s^2d^6	Co 27 s^2d^7	Ni 28 s^2d^8	Cu 29 s^2d^9	Zn 30 s^2d^{10}	Ga 31 p	Ge 32 p^2	As 33 p^3	Se 34 p^4	Br 35 p^5	Kr 36 p^6
V O	Rb 37 s	Sr 38 s^2	Yt 39 s^2d	Zr 40 s^2d^2	Cb 41 sd^4	Mo 42 sd^5	Ma 43 sd^6	Ru 44 sd^7	Rh 45 sd^8	Pd 46 s^2d^{10}	Ag 47 s	Cd 48 s^2	In 49 p	Sn 50 p^2	Sb 51 p^3	Te 52 p^4	I 53 p^5	Xe 54 p^6
VI P	Cs 55 s	Ba 56 s^2	Rare Earths	Hf 72 s^2d^2	Ta 73 s^2d^3	W 74 s^2d^4	Re 75 s^2d^5	Os 76 s^2d^8	Ir 77 s^2d^9	Pt 78 s^2d^{10}	Au 79 s	Hg 80 s^2	Tl 81 p	Pb 82 p^2	Bi 83 p^3	Po 84 p^4	— 85 p^5	Rn 86 p^6
VII Q	— 87 s	Ra 88 s^2	Ac 89 s^2d	Th 90 s^2d^2	Ux_2 91 s^2d^3	U 92 s^2d^4												

RARE EARTHS

La 37 d	Ce 38 df	Pr 59 df^2	Nd 60 df^3	Il 61 df^4	Sm 62 df^5	Eu 63 df^6	Gd 64 df^7	Tb 65 df^8	Dy 66 df^9	Ho 67 df^{10}	Er 68 df^{11}	Tm 69 df^{12}	Yb 70 df^{13}	Lu 71 df^{14}

FIG. 2.—Periodic Table According to Atomic Numbers.

The letters s, p, d, etc., refer to types of electrons in valence shell, and the exponent gives the number of electrons of a particular type. The letters in the extreme left hand column refer to the type of X-ray level corresponding to the valence shell in the period.

The symbols, s, p, d, etc., refer to the designation for the various types of orbits, which have been derived from optical spectra.

Figure 2 shows the periodic arrangement of the elements with atomic numbers and the type of electrons which constitute the valence group.

Mention has been made above of the periodic recurrence of similar physical properties, and what interests us in connection with

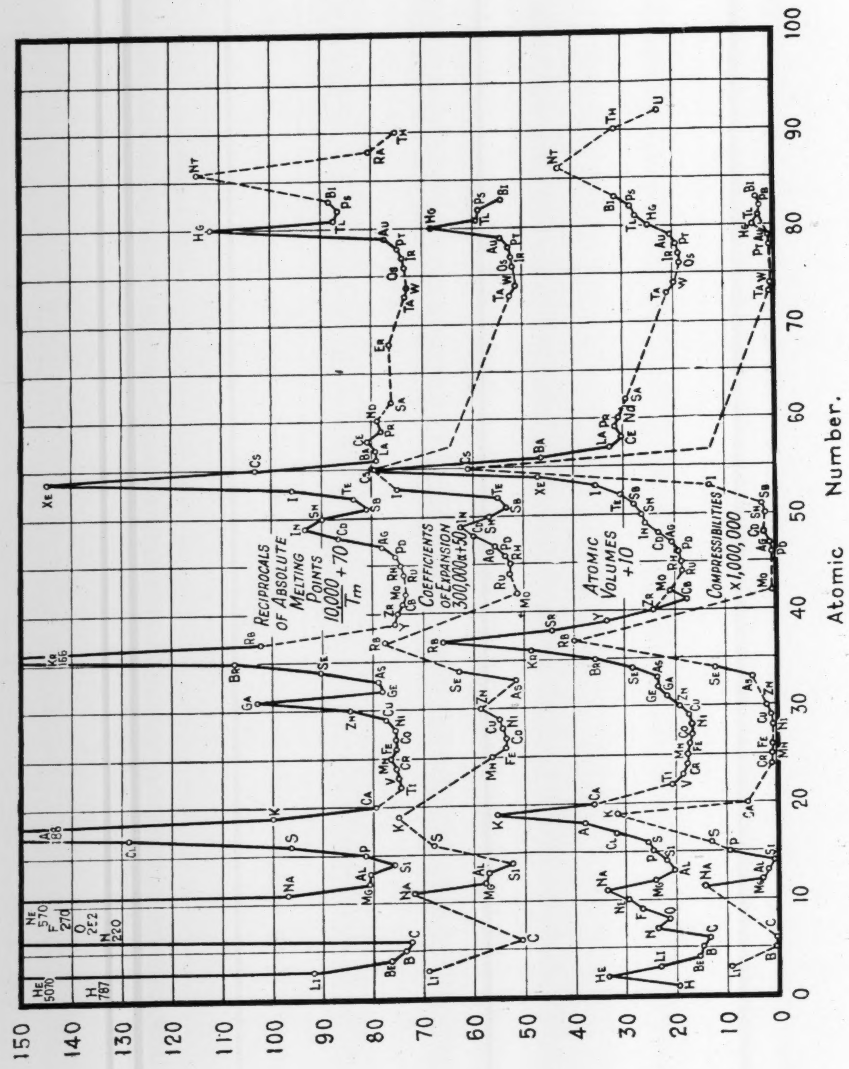


FIG. 3.—Four Cohesional Properties of the Elements as a Periodic Property of the Atomic Number.

the present discussion is the problem of cohesional properties. It is, of course, a well-known fact that the strength of materials as determined by the technical engineer depends not only upon the chemical composition of the material, but also upon the method of treatment, especially in the case of metals. It is, however, of importance to note that the most important cohesional properties of pure metals are evidently determined by the actual electronic arrangement in the atoms, since they are periodic properties of the atomic number. I shall therefore discuss these first, leaving for a subsequent section the discussion of those observations on strength of materials which at first glance seem to be governed by other factors than those of atomic structure.

Figure 3, taken from a paper by W. D. Harkins,[1] shows the variation with atomic number in four cohesional properties of the elements. It will be observed that those metals which are regarded as "strongest" from a technical point of view are characterized by:

1. High melting point, that is, low value of $1/T$,
2. Small value of the coefficient of expansion,
3. Low atomic volume, and
4. Low value of the coefficient of compressibility.

When we turn to a consideration of the corresponding atomic structures, we find that the metals showing maximum values of cohesional forces, such as chromium, molybdenum, tungsten, iron, nickel, palladium, platinum and similar elements, have incomplete *d* shells and one or two electrons in the next higher *s* shell. As shown in Fig. 2, these elements occur in the central groups of the periodic arrangement, that is, in Groups VI*a*, VII*a*, VIII and I*b*. The optical spectra of a number of these elements are extremely complex due to the fact that the valence electrons, that is, those present in incompleted groups, are able to form different configurations for which the energy differences are very slight. Thus for platinum, three different configurations are possible, and at present it is uncertain which corresponds to the normal state of the atom. It should be observed that the arrangements indicated in Fig. 2 have been derived from investigations of the spectra of the atoms at high temperatures, such as occur in arcs. It is, therefore, very probable that for the atoms in the solid state the distribution of electrons between *d* and *s* shells may be different. Thus the chromium atom may have the electron configuration d^6 or d^4s^2 instead of d^5s as shown. The variable chemical valencies of these elements also indicate the probability of these views.

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[1] W. D. Harkins and R. E. Hall, "The Periodic System and the Properties of the Elements," *Journal*, Am. Chemical Soc., Vol. 38, p. 169 (1916).

The fact that the atomic volumes of these elements, as well as their compressibilities, are low, leads to the conclusion that the cohesional forces between neighboring atoms must be very high.

On the other hand, the alkali metals which have a rather loosely bound single electron, as shown by relatively low ionizing energy, are also the most compressible of all the elements, and have the largest atomic volumes. The cohesional forces are therefore relatively low in these metals.

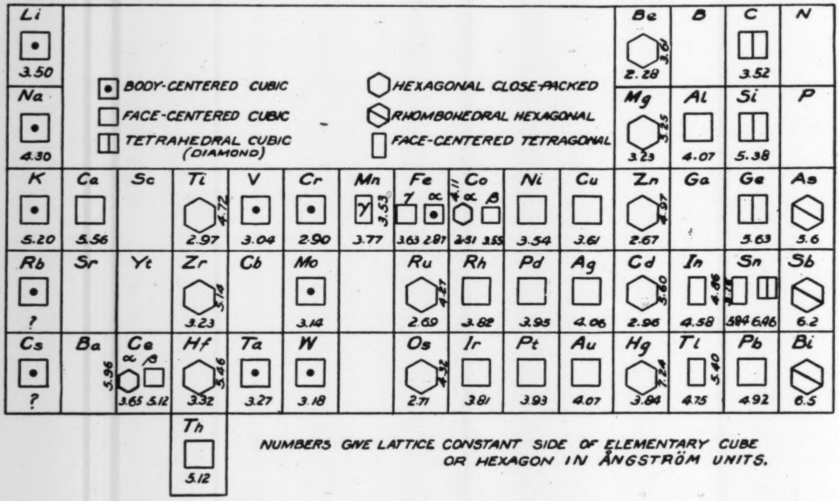
Li 3.50
Na 4.30
BODY-CENTERED CUBIC
FACE-CENTERED CUBIC
TETRAHEDRAL CUBIC (DIAMOND)
HEXAGONAL CLOSE-PACKED
RHOMBOHEDRAL HEXAGONAL
FACE-CENTERED TETRAGONAL
Be 3.61 2.28
B
C 3.52
N
Mg 3.25 3.23
Al 4.07
Si 5.38
P
K 5.20
Ca 5.56
Sc
Ti 4.72 2.97
V 3.04
Cr 2.90
Mn γ 3.53 3.77
Fe γ α 3.63 2.87
Co 4.11 α β 2.51 3.55
Ni 3.54
Cu 3.61
Zn 4.97 2.67
Ga
Ge 5.63
As 5.6
Rb ?
Sr
Yt
Zr 5.14 3.23
Cb
Mo 3.14
Ru 4.27 2.69
Rh 3.82
Pd 3.95
Ag 4.06
Cd 5.60 2.96
In 4.86 4.58
Sn 5.84 6.46
Sb 6.2
Cs ?
Ba
Ce α β 5.96 3.65 5.12
Hf 5.46 3.32
Ta 3.27
W 3.18
Os 4.32 2.71
Ir 3.81
Pt 3.93
Au 4.07
Hg 7.24 3.84
Tl 5.40 4.75
Pb 4.92
Bi 6.5
Th 5.12
NUMBERS GIVE LATTICE CONSTANT SIDE OF ELEMENTARY CUBE OR HEXAGON IN ÅNGSTRÖM UNITS.

FIG. 4.—Crystal Structures of Elements Arranged According to Periodic System.

When it is remembered that the technical strength of metals is largely dependent upon other factors, such as grain size, state of deformation, etc., it is remarkable that there are certain definite properties such as those shown in Fig. 3, which are independent of the ordinary mechanical properties. This fact in itself is sufficient to indicate that the latter are governed to a great extent by other factors than those due to interatomic forces, although it must be concluded that even these more complex factors must, in the last analysis, be correlated somehow with atomic structures. However, at the present time, we are far from having attained satisfactory conceptions in this regard for explanation of the observations summarized in the curves of Fig. 3.

EVIDENCE OF CRYSTAL STRUCTURE

Atoms and molecules are capable, under definite conditions, of forming regular lattice structures which constitute crystals, and, as might be expected, the evidence regarding crystal structures of the elements is in accord with expectations based on the periodic arrangements and the electronic configurations of the atoms. Atoms which belong to the same group crystallize in the same lattice type as shown in Fig. 4. The figures attached to each element give the lattice constants in Ångström units (1Å. = 10^{-8} cm.). Table II gives the same data arranged according to type of crystal structure and contains values of the lattice constant *a*, and minimum distance, *d*, between atoms in the crystal lattice for various metallic elements.[1,2]

TABLE II.—CRYSTAL STRUCTURE, LATTICE CONSTANTS, *a*, OF METALLIC ELEMENTS, AND MINIMUM DISTANCE OF APPROACH BETWEEN ATOMS, *d*, IN CRYSTAL LATTICE.

FACE-CENTERED CUBIC LATTICE

Metal	Aluminum	Calcium	Iron-γ	Cobalt-β	Nickel-α	Copper	Palladium
$a \times 10^8$	4.046	5.56	3.63	3.55	3.54	3.597	3.859
$d \times 10^8$	2.860	3.97	2.57	2.51	2.50	2.54	2.73

Metal	Silver	Iridium	Platinum	Gold	Lead	Thorium
$a \times 10^8$	4.079	3.823	3.912	4.065	4.920	5.04
$d \times 10^8$	2.88	2.70	2.77	2.88	3.48	3.56

BODY-CENTERED CUBIC LATTICE

Metal	Lithium	Sodium	Potassium	Vanadium	Chromium	Iron-α	Molybdenum	Tantalum	Tungsten
$a \times 10^8$	3.50	4.30	5.20	3.04	2.895	2.87	3.142	3.272	3.155
$d \times 10^8$	3.03	3.72	4.50	2.63	2.51	2.49	2.72	2.83	2.73

HEXAGONAL CLOSE PACKING

Metal	Beryllium	Magnesium	Titanium	Zinc	Zirconium	Cadmium	Cerium-α	Ruthenium	Osmium
$a \times 10^8$	2.283	3.22	2.97	2.670	3.23	2.960	3.65	2.686	2.714
$c \times 10^8$	3.607	5.23	4.72	4.966	5.14	5.632	5.96	4.272	4.32

For the hexagonal close packing, *a* corresponds to one parameter of the lattice, while *c* corresponds to the other. Diamond, silicon and gray tin crystallize in the cubical system, each atom being surrounded by four others equi-distant from each other, and located at the four corners of a regular tetrahedron.

The arrangement of atoms in a face-centered cubic lattice is "the same as that obtained when round shot are piled in layers as

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[1] P. P. Ewald, "The Structure of Solid Matter," *Handbuch der Physik*, Vol. 24, Chapter 4, p. 191, Julius Springer, Berlin, 1927.

[2] G. Sachs, "Mechanische Technologie der Metalle," Leipzig, 1925.

regularly and compactly as possible. In this structure each atom is equi-distant from twelve adjacent atoms." The hexagonal close packing represents an alternative form for the densest packing of spheres, while the body-centered cubic lattice is next in order of denseness of packing.

As emphasized by A. W. Hull:[1]

> Fifteen elements have their atoms arranged in the face-centered or cubic close-packed arrangement, and it is worthy of note that they are all metals and all ductile. The ductility may reasonably be ascribed to the arrangement, and one may generalize tentatively as follows. Any substance which crystallizes face-centered cubic will be ductile.[2]
>
> For the purpose of visualizing this relation, the type of lattice may be connected with the shape of the atom. The tendency of the atoms of certain substances to arrange themselves like closely packed solid spheres can best be pictured as due to the fact that the electrons in their outer shells are so symmetrically distributed and so closely coupled together as to present toward other atoms an essentially rigid spherical shell, with very weak residual localized forces. These atoms are held together in the solid crystalline state, not by the sharing of electrons or secondary valence forces, but by free electrons wandering at random between the atoms, occupying on the average fairly definite positions. These free electrons are the electrons that are superfluous to the formation of the stable spherical shells; for example, in copper, silver and gold, the single electron which is so easily lost in chemical reactions is certainly less firmly bound than the others, and must be superfluous to the formation of a stable shell. These elements will have one free electron for every atom, and the free electrons will occupy, on the average, the positions of the chlorine ions in sodium chloride. In magnesium zinc and cadmium there are presumably two free electrons per atom, with average positions the same as the fluorine atoms in CaF_2.

In the case of substances like $CaCO_3$, $ZrSiO_4$, $NaNO_3$, metallic oxides and sulfides, the atoms are also found to be arranged in definite lattice structures, and except for the fact that the different atoms occur in the structure in the ratios demanded by the chemical formulas, it is impossible to associate an atom of one kind with a definite atom of another kind. In other words, the distinction between the chemical molecules disappears. The whole crystal is one large molecule, and it is necessary to revise our point of view regarding the nature of the forces between the atoms in the case of a crystal lattice. The following quotations from a most noteworthy paper on this subject by I. Langmuir[3] state his views on the nature of the cohesive forces in crystals:

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[1] A. W. Hull, "The Crystal Structures of the Common Elements," *Journal*, Franklin Inst., February, 1922, p. 189.

[2] A more accurate statement would be that "any metal which crystallizes face-centered cubic will be ductile."

[3] I. Langmuir, "The Constitution and Fundamental Properties of Solids and Liquids," *Journal*, Am. Chemical Soc., Vol. 38, p. 2221 (1916).

Two facts stand out clearly as a result of the consideration of these crystal structures. In the first place, it is evident that crystals are built up of atoms in such a way that each atom is chemically combined to all the adjacent atoms, while these in turn are combined to those beyond.

Secondly, we see that the arrangement of the atoms in general does not follow the usual rules of valency, but that each atom is combined with a much larger number of atoms than correspond to its normal valence.

In the past it has been customary to consider that solids and liquids are held together by "forces of cohesion" and to call these "physical forces" as distinguished from "chemical forces. . . ." There is no present justification for this distinction between chemical and physical forces. Cohesion, adsorption and surface tension are all manifestations of forces similar in their nature to those acting between the atoms of solid bodies. It is therefore advantageous to look upon these forces as direct results of chemical affinity. In this way it becomes possible to correlate these so-called physical phenomena with the known chemical characteristics of the atoms and groups of atoms forming the bodies.

Langmuir is thus led to ascribe the forces holding together the atoms in a crystal of a pure metal to the action of secondary or residual valences. Now it has seemed to the writer that these views are quite in accord with the most recent conclusions which have been deduced by London and Heitler as a result of the application of wave mechanics to the problem of chemical valence.[1] It is, of course, beyond the scope of the present paper to discuss this new theory of atomic mechanics; but perhaps it is possible to give the reader some conception of those aspects of the theory which bear more directly on the problem under consideration.

The most essential feature of the new mechanics is that it replaces the Bohr conception of definite electronic orbits by the conception that for any given point in the approximate space occupied by an atom there exists a certain probability for locating the electron at that particular point at a given instant of time. Thus if we consider, say, an atom of normal hydrogen and regard the nucleus as a fixed center of reference, it is no longer correct to describe the electron as occupying a circular orbit at a distance $r = 0.529 \times 10^{-8}$ cm. as was derived on the basis of Bohr's theory. The correct description of the position of the electron is rather as follows: There is a very high probability that the electron will be found at this distance, but there is also a certain probability that the electron will be located closer to the nucleus at a certain instant, and another, still smaller probability that it will be located further out. In other words, we describe the position of the orbit by means of a probability function such as that shown in Fig. 5 in which the magnitude of the ordinate corresponds to the probability, P, of locating the electron at the value of r given by the corresponding distance on the axis of abscissas. It will be

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[1] The new wave mechanics has been discussed in a more popular manner in a number of recent papers. See especially the paper by J. H. Van Vleck, "The New Quantum Mechanics," *Chemical Review*, Vol. 5, p. 467 (1928), which also contains references to other papers on this subject.

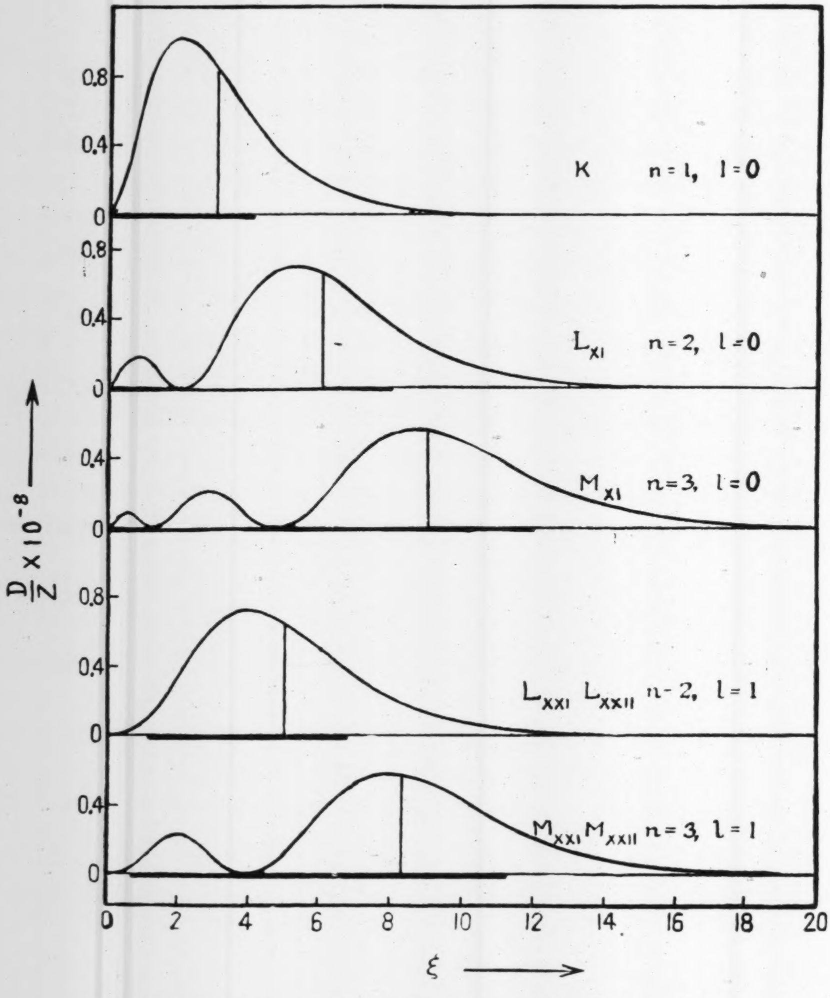

FIG. 5.—Electron Distribution for Different States of Hydrogen Atom, as a Function of Distance from Nucleus.

The ordinate gives the probability of occurrence of electron at distance given by abscissa. The type of electron is indicated by the capital letters *K*, *L*, etc., or the corresponding values of n and l.

observed that while the maximum value of P occurs at approximately the value $r = r_0$ given by the classical or Bohr theory, there is also a finite value of P for $r = 4r_0$ and for values of r beyond this.

In the case of a hydrogen molecule we have two electrons and two nuclei and the mathematical expression for the probability distribution of the two electrons becomes more complex. But as was shown by Heisenberg, the new wave mechanics leads to the conclusion that there will be a strong probability that the two electrons will be constantly interchanging places. Thus if we denote one nucleus by *a* and the other by *b*, and the corresponding electrons in the separate atoms by *1* and *2*, respectively, then when the two atoms are combined to form a molecule, there exists a certain probability that electron *2* will go to *a*, while electron *1* simultaneously goes to *b*. It is this interchange of electrons that is the equivalent of the valence bond in the old-fashioned chemistry.

Now if we extend this conception to atoms in a crystal which, according to Langmuir, are held together by secondary valence forces, we obtain not only an explanation of the cohesive forces between atoms in crystals in general, but also of the observed fact that certain elements possess these cohesive properties to a more marked degree than others. Referring again to Fig. 2, it is observed that elements like iron, cobalt, nickel, tungsten, platinum, etc., are characterized by the presence of large numbers of electrons in incompleted *d* groups. In a crystal lattice of any of these metals, there will be a constant interchange of electrons between neighboring atoms, thus leading to the same kind of cohesion as exists between two hydrogen atoms in a molecule of H_2.

According to this view, the magnitude of the cohesion will depend to a large extent on the frequency of the interchanges per electron and probably to a lesser extent on the number of electrons in the incompleted group. While it must be left to the theoretical physicists to develop these views quantitatively, we may deduce from analogy with the case of the hydrogen molecule (as treated by London and Heitler) that the frequency of interchange will be given by the *heat of evaporation*. We are thus led to the conclusion that the metals showing highest cohesion should be those which have the highest heat of evaporation. Thus tungsten and carbon which have the highest heats of evaporation of all the elements are certainly also the most remarkable metals in regard to magnitude of cohesion in the solid state.

On the basis of the quantum theory, the frequency, ν, of interchange per electron is given by the relation:

$$\nu = \frac{L_0}{Nh}$$

where L_0 = latent heat of evaporation at $T = 0$,
N = number of atoms per gram atom = 6.062×10^{23},
h = the quantum constant = 6.55×10^{27} erg seconds.

Expressing L_0 in calories per gram atom, and h in the same units, this relation assumes the form:

$$\nu = 1.048 \times 10^{10} L_0$$

Table III gives the values of L_0 for various metals together with the calculated values of ν.[1, 2]

The fourth column gives values of κ, the coefficient of compressibility; and the fifth column, the values of α, the coefficient of expansion. The parallelism between the values of L_0 (or ν), κ and α is very striking and indicates that the metals exhibiting highest cohesion according to the criterion of heat of evaporation also have the smallest values of the coefficients of compressibility and expansion. Further inferences which may be drawn from these data are discussed in a subsequent section.

TABLE III.—LATENT HEAT OF EVAPORATION, L_0. DERIVED FREQUENCY OF INTERCHANGE OF ELECTRONS, ν, AND COEFFICIENTS OF COMPRESSIBILITY, κ, AND EXPANSION, α, FOR SEVERAL METALLIC ELEMENTS.

METAL	LATENT HEAT OF EVAPORATION, L_0, CALORIES PER GRAM ATOM	DERIVED FREQUENCY OF INTERCHANGE, $\nu \times 10^{-15}$	COEFFICIENT OF COMPRESSIBILITY, $\kappa \times 10^{12}$	COEFFICIENT OF EXPANSION, $\alpha \times 10^6$
Tungsten	191 880	2.01	0.303	13
Molybdenum	146 000	1.53	0.358	16
Platinum	127 500	1.34	0.372	27
Nickel	89 440	0.936	0.542	40
Iron	89 025	0.933	0.606	36
Copper	82 060	0.860	0.756	50
Silver	71 320	0.747	1.020	59
Zinc	31 330	0.328	1.76	106
Cadmium	26 770	0.427	2.27	114

It is worth noting that the values of ν as calculated from the heat of evaporation are of approximately the same order of magnitude as the reciprocal of a magnitude which is known as the "time of

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[1] The values of L_0 for the metals tungsten to silver are taken from the paper by H. A. Jones, I. Langmuir and G. M. J. Mackay, "The Rates of Evaporation and the Vapor Pressures of W, Mo, Pt, Ni, Fe, Cu and Ag," *Physical Review*, Vol. 30, p. 201 (1927). The values for zinc and cadmium are from the paper by A. C. Egerton, "The Vapor Pressure of Zinc, Cadmium and Mercury," *Philosophical Magazine*, Vol. 33, p. 33 (1917).

[2] Values for κ, the coefficient of compressibility and α, the coefficient of cubical expansion are from International Critical Tables, Vol. 3, p. 46, and Vol. 2, p. 459, respectively.

relaxation." The definition of this quantity is given by Langmuir[1] as follows:

> Let us imagine all the atoms in the surface layer of a solid to have their temperature suddenly raised by a given amount. This layer will then rapidly cool and will approach the temperature of the underlying layers. The rapidity with which this temperature equalization takes place will measure the rate at which a motion of one atom affects the adjacent ones. A simple approximate calculation gives for the time necessary for the temperature increment of the surface layer to fall to 37 per cent $\left(\frac{1}{e}\text{th}\right)$ of its initial value:
>
> $$t_r = \frac{3k}{H\sigma}$$
>
> Here k is the Boltzmann gas constant (1.372×10^{-16} ergs per degree), H is the heat conductivity of the solid, and σ is the distance between adjacent atoms.

The period t_r is known as the "time of relaxation." For copper and platinum the values of H at room temperature in ergs per second per sq. cm. are 3.88×10^7 and 0.695×10^7, respectively, while for σ we may use the values 2.54×10^{-8} and 2.77×10^{-8} cm. respectively. Hence we obtain for t_r the values 4.18×10^{-16} and 2.14×10^{-15} for copper and platinum, respectively. The reciprocals of these periods are 2.39×10^{15} and 4.68×10^{14} per second respectively, which are of the same order of magnitude as the values of ν calculated from the latent heat of evaporation.

This result is to be expected qualitatively on the basis of the electron theory of conduction of heat and electricity in metals, for the frequency of interchange of electrons between adjacent atoms must affect both the thermal and electrical conductivity. However, it would take us much beyond the scope of the present paper to discuss this point at further length.

THEORY OF COHESION IN SOLIDS

While it has not been found possible as yet to formulate a quantitative theory of cohesion in solids on the basis of atomic structure, it is possible to deduce relations between the cohesive forces and such experimentally measured quantities as the latent heat of evaporation and the coefficients of thermal expansion and compressibility, by introducing very plausible assumptions regarding the laws governing attractive and repulsive forces between atoms.

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[1] I. Langmuir, "The Constitution and Fundamental Properties of Solids and Liquids," *Journal*, Am. Chemical Soc., Vol. 38, p. 2221 (1916).

We shall first of all discuss the theory developed mainly by Grüneisen for the case of monatomic pure metals.[1, 2, 3] Let us consider a crystal of a metal such as copper or silver. As has been mentioned previously, it has been shown by crystal analysis that the atoms are arranged according to a simple face-centered lattice with single atoms at each point of the lattice. On the basis of the kinetic theory, it follows, furthermore, that the atoms vibrate about certain positions of equilibrium under the action of attractive and repulsive forces between them, and with frequencies of the order of 10^{12} per second, which can be calculated by several different methods.

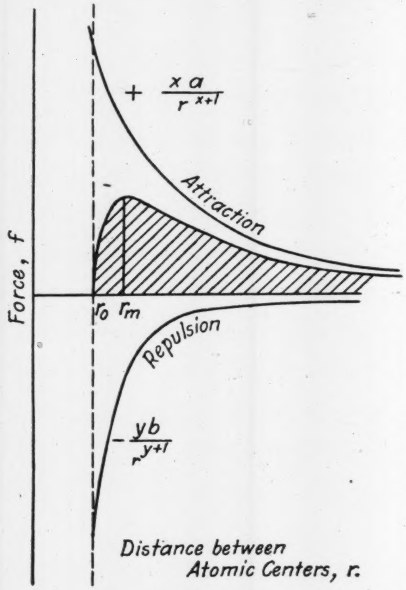

FIG. 6.—Diagram Illustrating Laws for Variation with Distance, r, Between Atoms, of Attractive and Repulsive Forces.

The shaded area represents the energy necessary to separate the two atoms completely (heat of evaporation).

We know that at least at ordinary temperatures the atomic heat (specific heat × atomic weight) at constant volume is approximately 6.0 calories for most of the metallic elements. This has been ex-

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[1] E. Grüneisen, "Theory of the Solid State of Monatomic Elements," *Annalen der Physik*, Vol. 39, p. 257 (1912). This theory is also discussed by J. K. Roberts, "Heat and Thermodynamics," Chapter XXII, Blackie and Son, Ltd., London, 1928; also G. Heckmann, "The Lattice Theory of Solids," *Ergebnisse der exakten Naturwissenschaften*, Vol. 4, p. 100 (1925).

[2] M. Born and O. F. Bollnow, "Structure of Solid Materials, Theoretical Foundations," *Handbuch der Physik*, Vol. XXIV, Chapter V.

[3] A. F. Joffé, "The Physics of Crystals," McGraw-Hill Book Co., 1928. See especially Lecture II for discussion of theory of crystal lattice.

plained by assuming that the thermal energy taken up by the atoms must be distributed in such a way that it is divided equally between kinetic and potential energy of vibration. Since according to a well-known fundamental law of physics (the principle of equi-partition of energy), the increase in vibrational kinetic energy per gram atom per 1° C. must be equal to $\frac{1}{2} R = \frac{1}{2} \times 1.986$ calories, per degree of freedom, it follows that the total atomic heat (kinetic + potential) must be $2 \times \frac{3}{2} \times 1.986 = 5.96$ calories per gram atom for an atom executing harmonic vibration about a mean position of equilibrium. (Since this vibration has three components, one along each of the three axes of coordinates required to describe the vibration, the factor 3 is necessary.) "The fact that the observed atomic heat is 5.96 calories may be taken as proof that the forces to which the atoms are subjected vary approximately in proportion to the displacement of the atoms from equilibrium positions. There must then be both attractive and repulsive forces acting between the atoms. On the average these opposing forces must just balance each other. As one atom approaches another the repulsive force must gradually increase and the attractive force decrease until the repulsive force greatly predominates. We cannot consider that the repulsive forces in solids are exerted only during collisions between atoms, for under these conditions there would be no potential energy, and the atomic heat would be $\frac{3}{2} \times 1.986$."[1]

As has been stated by Joffé:[2] "It is obvious that, for stable equilibrium in an expansion of the crystal as a whole, the attractive forces must predominate between the units, while in compression the repulsive forces between the units must predominate. Thus, the repulsion must decrease more rapidly than attraction when the distance is increased, and must increase more rapidly than the attraction when the distance is decreased."

We are thus led to regard the total energy for a combination of two atoms as made up of two terms, one for attraction and the other for repulsion in accordance with a relation of the form

$$U = -\frac{a}{r^x} + \frac{b}{r^y} \dots\dots\dots\dots\dots\dots\dots (1)$$

where U is the energy per gram atom, a and b are constants in the terms for attraction and repulsion respectively, r is the distance between the atomic centers, and x and y are definite exponents whose values are to be determined.

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[1] See footnote 1, p. 23.

[2] See footnote 3, p. 24.

At the normal distance, r_0, the forces of attraction and repulsion are in equilibrium. Consequently the energy U must be a minimum. Hence follow the relations:

$$\frac{dU}{dr} = 0$$

and

$$\frac{xa}{r_0^{\,x+1}} = \frac{yb}{r_0^{\,y+1}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

or

$$yb = xa\, r_0^{\,(y-x)}$$

Figure 6 shows the manner in which the values of the forces $\frac{xa}{r^{x+1}}$ and $\frac{yb}{r^{y+1}}$ vary with r for certain assumed values of x and y. It should be observed that from what has been stated previously regarding the variation in these forces with distance it follows that the exponent y is greater than x.

That is, the force of repulsion decreases more rapidly with increase in r than the force of attraction. This is well illustrated in the figure, where the upper curve gives the force of attraction, the lowest curve that of repulsion, both as functions of r, while the middle curve shows the resultant or net cohesive force. It is seen that for $r = r_0$ this is zero, while it passes through a maximum at a certain value $r = r_m$. The total area under the middle curve represents the energy required to separate the two atoms completely.

It is evident that for any substance there must exist some sort of relation between the magnitudes of the exponents x, y, the constants a and b and the observed magnitudes of the latent heat of vaporization and coefficients of compressibility and thermal expansion. We shall now proceed to follow Grüneisen in the derivation of such a series of relations.

For this purpose we shall replace the exponent x in the second term of Eq. 2 by 3, for the force of attraction between atoms which are uncharged must vary more rapidly than according to the inverse square law and the choice of the value $x + 1 = 4$ is found to be in best agreement with all that we know about the cohesive properties of metals. It is furthermore obvious that in calculating the total energy per gram atom, by summing up a series of expressions such as that given in Eq. 2, each atom will be counted twice. We thus obtain for the total energy per gram atom at the absolute zero, the relation:

$$U_o = -\tfrac{1}{2}N\frac{a}{r^3} + \tfrac{1}{2}N\frac{b}{r^y}$$

$$= -\frac{A}{v_o} + \frac{B}{{v_o}^m} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

where $\frac{1}{2}aN^2 = A$, $\frac{1}{2}bN^{\frac{y}{3}+1} = B$, and $\frac{y}{3} = m$,

while $N = 6.062 \times 10^{23}$ = number of atoms per gram atom.

At $T = 0$,

$$\frac{\delta U_o}{\delta v_o} = 0$$

Hence

$$\frac{A}{v_o} = \frac{mB}{{v_o}^m}, \quad \text{and } B = \frac{A\,{v_o}^{m-1}}{m} \quad \ldots\ldots\ldots\ldots (4)$$

At higher temperatures, the total energy is increased by that due to atomic vibration and the relation for the energy, U, becomes

$$U = -\frac{A}{v} + \frac{B}{v^m} + \tfrac{1}{2}\int_0^T C_v dT$$

where C_v = specific heat per gram atom at constant volume.

From this relation and a consideration of the vibrational motions of the atoms, Grüneisen finally derives an *equation of state for a solid* of the form:

$$pv + \frac{A}{v} - \frac{mB}{v^m} = \gamma\int_0^T C_v dT \quad \ldots\ldots\ldots\ldots (5)$$

where $\gamma = \frac{3m+2}{6}$, and therefore $m = 2\gamma - \frac{2}{3}$. . . . . (6)

Now from thermodynamical considerations, it follows that

$$\gamma = -v\left(\frac{\delta v}{\delta T}\right)_p \Big/ C_v\left(\frac{\delta v}{\delta p}\right)_T = -\frac{v\alpha_p}{C_v\kappa_T} \quad \ldots\ldots\ldots\ldots (7)$$

where α_p = coefficient of cubical expansion at constant pressure,
κ_T = coefficient of compressibility at constant temperature, and
v = atomic volume.

Hence we can calculate the values of γ and consequently those of m from experimentally observed values of α_p, κ_T and v.

Again, the total energy at $T = 0$ corresponds to the latent heat of vaporization at this temperature, λ_o, in accordance with the relation

$$U_o = -\lambda_o \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

It therefore follows from Eqs. 4 and 8 that

$$\lambda_o = \frac{A}{v_o}\left(1 - \frac{1}{m}\right)$$

or

$$A = \frac{\lambda_o v_o m}{m - 1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

from which A can be calculated for corresponding values of λ_o, m and v_o.

The energy due to attractive forces is

$$\Phi_o = -\frac{A}{N^2 r^3}$$

Hence F, the force of attraction on an atom is given by

$$F = \frac{d\Phi_o}{dr} = \frac{3A}{N^2 r^4}.$$

The number of atoms per square centimeter is approximately $\frac{1}{4r^2}$. Hence the force per unit area, or *cohesive force*, is given by

$$\left.\begin{aligned} P_c &= \frac{3A}{4N^2 r^6} = \frac{3A}{4v_o^2} \text{ dynes per sq. cm.} \\ &= \frac{3A}{4v_o^2 \times 68{,}947} \text{ lb. per sq. in.} \ldots\ldots \end{aligned}\right\} \ldots\ldots\ldots (10)$$

Table IV gives values of m, A and P_c as calculated from the above equations for those metals for which reliable values of λ_o are available. The values of λ_o, α and κ used in the present case are those already given in Table III.

It will be observed that the value of the exponent m lies between 2.5 and 5 approximately, and since $y = 3m$, this corresponds to a force of repulsion which varies inversely with the 8.5 to 16th power of the distance; while the force of attraction varies inversely as the 4th power of the distance.

The values of P_c given in the last two columns are of special interest since they lead to the conclusion that the *theoretical or "molecular" cohesive strength of metals is between 10 and 100 times the technical rupture strength.*

A similar conclusion has also been deduced by I. Traube[1] by applying an equation derived by van der Waal for the relation between the attractive force between molecules and the latent heat of vaporization.

He has also pointed out that in general the order of the metals with decrease in theoretical cohesive strength is the same as that for decrease in both Young's modulus of elasticity and hardness as measured on Mohr's scale.

Before passing on to the discussion of the explanation which has been offered of the existence of this discrepancy between molecular and technical strength, it is also important to consider the theory of cohesion in ionic lattices as developed mainly by M. Born.[2] A typical example of such a lattice is that of rock salt, NaCl, in which positive and negative ions are arranged at alternate corners of a simple cubical lattice structure. Each ion vibrates about a mean position of equilibrium under the action of attractive and repulsive forces. In this case the former is assumed to be the Coulomb force which varies directly as the product of the two electric charges and inversely as the square of the distance, while the force of repulsion varies inversely as r^{n+1} where n is greater than 2. Thus the energy, U, of the lattice, per gram molecule is expressed by a relation of the form:

Table IV.—Molecular Cohesive Forces Calculated by Grüneisen's Theory from Coefficients of Cubical Expansion and Compressibility.

Metal	Atomic Volume, v, cu. cm.	m	$A \times 10^{-13}$, dynes-centimeters	Cohesive Force, P_c, $\times 10^{-11}$, dynes per sq. cm.	$\times 10^{-6}$, lb. per sq. in.
Tungsten	9.7	2.66	12.48	9.95	14.45
Molybdenum	10.7	3.35	9.57	6.27	9.09
Platinum	9.1	4.59	6.21	5.62	8.16
Nickel	6.7	3.29	3.61	6.02	8.74
Iron	7.1	2.71	4.20	6.25	9.06
Copper	7.1	3.09	3.61	5.37	7.78
Silver	10.3	4.09	4.07	2.88	4.17
Zinc	9.2	3.77	1.64	1.46	2.11
Cadmium	13.1	4.60	1.88	0.82	1.19

$$U = -\frac{Ae^2}{r} + \frac{B}{r^n} \qquad (11)$$

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[1] I. Traube, "Physical Properties of the Elements from the Standpoint of the van der Waal Equation," *Zeitschrift für anorganische Chemie*, Vol. 34, p. 413 (1903).

[2] See footnote 2, p. 24.

where A and B are constants similar to those used in Eq. 4 while e is the unit electric charge. For equilibrium at $T = 0$ it is necessary that

$$\frac{\delta U}{\delta r} = 0$$

and hence

$$r_0 = \left(\frac{A}{nB}\right)^{\frac{1}{n-1}} \quad \dots\dots\dots\dots\dots\dots\dots\dots (12)$$

where r_0 = the lattice constant $= \left(\frac{M}{N\rho}\right)^{\frac{1}{3}}$

M = molecular weight
ρ = density
$N = 6.062 \times 10^{23}$.

Similarly it may be shown that the volume compressibility, κ, is given by the relation

$$\frac{1}{\kappa} = \frac{(n-1)\, e^2 A}{9\, r_0^4} \quad \dots\dots\dots\dots\dots\dots\dots\dots (13)$$

while A may be calculated for any given lattice type, as has been shown by Madelung,[1] on the basis of certain theoretical considerations.

In this manner it has been found possible to compute n from the observed values of κ. For crystals of the alkali halides the calculated values of n vary between 6.1 for lithium iodide to 10.3 for cesium bromide.[1]

Using a mean value $n = 9$, Born has calculated the energy (in kilo-calories per gram molecule) for different types of lattice structures as follows:

Type NaCl.............................. $-U = 545\ (\rho/M)^{\frac{1}{3}}$
" CaF_2.............................. $-U = 1779\ (\rho/M)^{\frac{1}{3}}$
" ZnS.............................. $-U = 2120\ (\rho/M)^{\frac{1}{3}}$

The negative value of U corresponds to the energy required to convert the lattice into positive and negative ions in the vapor state, and may be calculated as shown by Born from the following thermal data:

1. The heat of formation of the salt from the elements, Q_{MX},
2. The heat of dissociation of the halogen, D_X,
3. The heat of sublimation of the metal atoms, S_M,
4. The energy of ionization of the metal atom, I_M, and
5. The electron affinity of the halogen atom, E_X.

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[1] See footnote 2, p. 24.

The relation between these five quantities and U_{MX}, the lattice energy, is given by a cycle process in the form:

$$Q_{MX} + U_{MX} + (I_M + S_M) + (D_X - E_X) = 0$$

The values of U, as derived from thermal data, are found to be in very good agreement with those derived from the theoretical considerations stated above, as shown in the following table:

	$-U$ OBSERVED, KILO-CALORIES	$-U$ CALCULATED, KILO-CALORIES
NaCl	183	182
NaBr	170	171
NaI	159	158
KCl	160	162
RbCl	161	155

The theoretical investigations of Born and others working with him have been extended by J. E. Lennard-Jones and his associates.[1] In the papers published by these investigators an attempt has been made to coordinate kinetic theory data obtained from a study of the behavior of the rare gases with the forces between ions of similar electronic structure. Thus Cl^- and K^+ have the same number of electrons as the atom of argon, while Na^+ and F^- are similarly related to neon. Thus Lennard-Jones and P. A. Taylor[1] have determined the laws governing the repulsion between ions and have used the conclusions to calculate a number of crystal properties including compressibility and elasticity, as well as surface energy. "The investigation indicates," according to their statement, "that the repulsive forces of the neon-like ions in crystals can be represented by an inverse eleventh power law, each ion having its appropriate force constant, while the repulsive forces of the argon-like ions can be represented by an inverse ninth power law. . . . The forces determined in this paper are found to be singularly successful in explaining observed interatomic distances in crystals, and are shown to lead to values of crystal energies and compressibilities which are in satisfactory agreement with observation."

The fact that these authors have found it possible to calculate values of Young's modulus for different ionic lattices and surface energies from data which are dependent in the last analysis upon observations of the viscosity and other properties of the rare gases—this fact is a striking evidence of the progress which has been made in at least one direction in coordinating cohesive properties with atomic structure.

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[1] J. E. Lennard-Jones and P. A. Taylor, "Some Theoretical Calculations of the Physical Properties of Certain Crystals," *Proceedings*, Royal Soc. of London, A, Vol. 109, p. 476 (1925); also J. E. Lennard-Jones and B. M. Dent, "The Forces Between Atoms and Ions," *Proceedings*, Royal Soc. of London, A, Vol. 112, p. 230 (1926).

The data on elastic moduli of salts are very meager, but the values calculated by Lennard-Jones and Taylor are certainly in fair agreement with such data as are available and the results must therefore be regarded as indicating that the assumptions of the theory are justifiable to a high degree.

In the case of the surface energy, the same investigators find that the ratio for the 011 and 100 planes is in agreement with the observations of Born and Steine. It is of interest to note that the values calculated for the surface energy of ionic crystals are quite large compared with those actually observed for the salts in the molten state. The following table gives calculated values of σ, the surface energy (in ergs per square centimeter) for some of the crystals:

CRYSTAL	SURFACE ENERGY, σ, FOR 100 PLANE, ERGS PER SQ. CM.	SURFACE ENERGY, σ, FOR 011 PLANE, ERGS PER SQ. CM.
NaF	304	784
NaCl	96	350
MgO	1362	3940
CaO	1032	2850
KCl	76.6	350

The observed values for σ in the molten state are 66.5 ergs and 69.3 ergs for NaCl and KCl, respectively.

TECHNICAL RUPTURE STRENGTH AND SURFACE ENERGY

From the values of U, the energy of the crystal lattice determined as already mentioned, the cohesive strength may be calculated as in the case of metallic lattices. In fact, the mechanical cohesion of the lattice and the chemical attraction of its ions are one and the same, as pointed out by Born. F. Zwicky[1] has carried out such a calculation and concludes that for rock salt the theoretical rupture strength should be $P_c = 2.10^{10}$ dynes per sq. cm. = 20,400 kg. per sq. cm. = 290,000 lb. per sq. in., while the observed value is:

$$\begin{aligned} R &= 6.31 \times 10^7 \text{ dynes per sq. cm.} \\ &= 64.4 \text{ kg. per sq. cm.} \end{aligned}$$

The existence of this discrepancy between technically measured rupture strength and theoretically calculated cohesive force has naturally presented a problem of great interest, and as a result of the work of A. A. Griffiths, A. Smekal, M. Polanyi, A. Joffe, F. Zwicky and others a very satisfactory solution has been deduced.

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[1] F. Zwicky, "The Rupture Strength of Rock Salt," *Physikalische Zeitschrift*, Vol. 24, p. 131 (1923).

Griffiths[1] found that very fine fibers of glass are very much stronger than large ones. Thus for 40-mil fibers the breaking stress was found to be 24,900 lb. per sq. in., and increased with decrease in diameter, finally reaching the value of approximately 500,000 lb. per sq. in. for 0.13-mil fibers.

Similar observations have been made on drawn wires of various metals. Thus the tensile strength of cold-drawn coppered steel wire is increased[2] from 128.0 to 207.6 kg. per sq. mm. as the diameter is decreased from 5.59 to 1.83 mm. and the following data (Table V) published by Z. Jeffries[3] for the tensile strength of tungsten show tremendous variation in strength with diameter.

TABLE V.—TENSILE STRENGTH OF TUNGSTEN.

KIND OF MATERIAL	DIAMETER, MILS.	TENSILE STRENGTH, LB. PER SQ. IN.
Sintered tungsten ingot	200 by 250	18 000
Swaged rod	216.0	50 600
Swaged rod	125.0	107 000
Swaged rod	80.0	176 600
Swaged rod	26.0	215 000
Drawn wire	18.0	264 000
Drawn wire	7.23	340 000
Drawn wire	5.78	366 000
Drawn wire	5.50	378 000
Drawn wire	3.96	483 000
Drawn wire	1.14	590 000
By extrapolation	0	800 000 (approx.)

By extrapolating his data to zero diameter, Griffiths concluded that the maximum strength for infinitesimally thin glass fibers would be 1.6×10^6 lb. per sq. in., which is of the same order of magnitude as the theoretically calculated cohesive strengths for metals given in Table IV. He therefore suggested that extremely fine cracks on the surface might be the real explanation of the observed rupture strengths. By application of a relatively low average stress, a much higher stress would be produced at the edges of the cracks and thus lead to rupture. Griffiths showed[4] that according to this point of view the rupture strength, S, should be related to the surface energy σ, and length of crack l by a relation of the form:

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[1] A. A. Griffiths, "The Phenomena of Rupture and Flow in Solids," *Philosophical Transactions*, Royal Soc. of London, A, Vol. 221, p. 163 (1921).

[2] International Critical Tables, Vol. 2, p. 509.

[3] Z. Jeffries, "Effect of Temperature, Deformation and Grain Size on the Mechanical Properties of Metals," Am. Inst. Mining and Metallurgical Engrs., Vol. 60, p. 474 (1919).

[4] A simple derivation of the relation between rupture strength and length of cracks is given by A. Smekal, "Technical Strength and Molecular Cohesion," *Naturwissenschaft*, Vol. 10, p. 799 (1922).

$$S^2l = k\sigma \text{..........................} (14)$$

where k is a proportionality constant whose value depends upon the values of Young's modulus and Poisson's ratio. That is, $S\sqrt{l}$ should be constant. This was confirmed by a series of experiments on the bursting strength of glass bulbs and tubes which had been scratched previously to produce fine cracks. Assuming that in these experiments the radius of curvature at the corners of the cracks was of the same order as molecular dimensions, Griffiths showed that the maximum stress at rupture was about 3×10^6 lb. per sq. in.

That in the case of rock salt the breaking strength actually observed does not correspond to the true cohesive strength, but is influenced by surface effects which are probably small cracks, was shown by Joffe and his associates.[1] By breaking the crystals under water so that the surface became free of small cracks and irregularities by the action of the solvent, it was possible to increase the rupture strength to 16,000 kg. per sq. cm., which is almost equal to the calculated value of 20,400 kg. per sq. cm.

Another method, by which the influence of surface crevices was avoided, and a more nearly correct measure of the true cohesive strength obtained, was the following: If a sphere of the material is heated suddenly and uniformly, the outside layers will expand first and stretch the central cold part of the sphere, exerting a uniform tension, which may be calculated. Spheres of rock salt were cooled in liquid air and then plunged suddenly into hot water at 100° C. and fused lead at 600° C. There was no cracking or rupture if precautions were taken to produce a uniform heating of the sphere. The maximum uniform tension at the center in these spheres must have been as high as 6000 kg. per sq. cm.

There has thus accumulated a great deal of experimental evidence that the technical strength of materials depends upon the presence of surface cracks or crevices or other kind of irregularities.

A. Smekal[2] elaborated this suggestion and has discussed his views on this subject in a number of papers. He remarks as follows:

> The only logical conclusion (to account for the difference between technical and theoretical rupture strength of materials) is the assumption that ordinary technical materials are far from being homogeneous, as we have always assumed, but that they have many fine, microscopic and submicroscopic holes, cracks and other inhomogeneities, which decrease the technical strengths as compared

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[1] See footnote 3, p. 24.

[2] See footnote 4, p. 33.

> with the molecular strength. The former is thus determined on the basis of a macroscopic stress distribution, behind which there is hidden a quite different sort of microscopic stress distribution which depends on the inhomogeneities present in the material. . . . The old experience that coarsely crystalline material possesses a lower (technical) strength than finely crystalline, thus receives a very obvious explanation.
>
> If this view is correct, the strength of any material should be considerably increased if care is taken to reduce to a minimum the occurrence of cracks and other inhomogeneities by choice of correspondingly small dimensions and careful thermal treatment of the body under test.. In fact it has been known for a long time, for example, that very thin wires are much stronger than thicker wires; in the ideal limiting case of a single chain of molecules, the rupture strength would become of the same magnitude as the molecular strength. . . .
>
> From the above remarks we must conclude that the technical strength does not represent, in general, a physical constant characteristic of a given material, whereby the *practical significance* (of such a value) is naturally not taken into consideration. The physicist requires for his purpose a modification of the material which is as free from flaws as possible and reproducible in every respect, and that is the *single crystal*. As a matter of fact, it is found that not only the rupture strength but also, as has been shown by G. V. Hevesy, the electrolytic as well as thermal conductivity is quite different in the polycrystalline state from those of single crystals.

Smekal is thus led to the conclusion that the rupture strength of a single crystal must increase with decrease in dimensions of the crystal until it approaches the molecular strength, and that for any given material there exist upper limits for these dimensions. F. Zwicky,[1] who has developed Smekal's ideas still further, has discussed this point of view in a very recent paper as follows:

> Smekal thinks that actual crystals are never of an ideal geometrical structure, but are made up of a great many microscopic blocks leaving cracks and other imperfections between them. We might call such a structure a mosaic crystal according to a term used by Ewald.[2] Smekal correlated this conception with a great many facts such as electric conductivity, electric strength, photoelectric absorption,[3] etc. He succeeded in determining the size of the elementary blocks in a half empirical way and he found that they contain in the case of rock salt and in many other cases about 10,000 atoms each. As Smekal's considerations enable us to understand a great number of facts, it has to be concluded that the formation of cracks and blocks cannot be of a purely accidental nature. Otherwise, the surface cracks should occur in all kinds of different sizes on different samples. But then, it is impossible to understand why the technical breaking strength is a relatively well-defined quantity, as the different samples of a crystal would break on application of widely varied specific stresses.

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[1] F. Zwicky, "On the Imperfections of Crystals," *Proceedings*, Nat. Academy Sciences, Vol. 15, p. 253 (1929).

[2] See footnote 1, p. 17.

[3] A. Smekal, "On the Molecular Theory of Strength and Hardening," *Zeitschrift für techn. Physik*, Vol. 7, p. 535 (1926); also, A. Smekal, "Electrical Conductivity and Dielectric Strength of Insulating Ionic Crystals," *Zeitscrift für techn. Physik*, Vol. 8, p. 561 (1927).

In order to account for this fact, Zwicky has pointed out that for a given crystal lattice there exists a definite maximum value of the lateral contraction, γ, on extension. For rock salt the value of γ is about 6 per cent.

> This means that considering the ions of one lattice plane only, they will try to arrange themselves in a lattice characterized by spacings which are 6 per cent smaller than the spacing in the solid crystal. This same tendency to contract is also exhibited by the surface planes, although in a lesser degree. . . . Now it is easily seen that the surface cannot contract as a whole, but will do so in patches of a definite average size between which cracks will be left open.

The size of the patches will be governed by the lattice constant and the depth of the cracks can be calculated from the magnitude of σ, the surface energy.

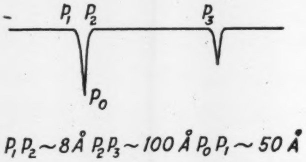

FIG. 7.—Illustrating Zwicky's Theory of the Formation of Ultra-Microscopic Surface Cracks (Mosaic Structure) at Intervals Governed by the Size of Unit Crystal Lattice.

As a result, the surface of a crystal will represent an appearance as approximately represented in Fig. 7. The dimensions given are those which we would have to expect for the largest cracks.

These are $P_1P_2 \sim 8$Å., $P_2P_3 \sim 100$Å., $P_0P_1 \sim 50$Å.

One further conclusion which can be drawn from the numerical relations given above is this. The crystal will not only crack on the surface, but also exhibit a great tendency for block formation all through its interior when not controlled properly during its growth. This follows from the fact that the energy gained from the contraction of a surface is of the same magnitude as the surface energy. It should be possible to avoid, at least partially, the formation of a mosaic structure in the crystal by growing it under very well-controlled conditions. But it is then to be expected from the theory that such crystals will be very unstable against mechanical and thermal disturbances. There are a number of facts known which seem to justify the above conclusions. A very instructive experiment is this. Certain organic crystals grown in the saturated solution are stable as long as they are entirely covered by the liquid. They break off immediately, however, as soon as the surface becomes dry. The fact that very thin glass and quartz fibers show a high strength just after formation, followed by a gradual decrease, might be interpreted in the same way. It is probable, also, that the instability of the property, known as cold hardening, falls into this category.

It can now be understood that the technical breaking strength will be a well-defined quantity. It will be determined by the size and shape of the largest cracks, which in turn may be obtained in principle by the above considerations.

COHESIVE PROPERTIES OF METALLIC SINGLE CRYSTALS

Since the crystal lattice forms the first unit in building up homogeneous materials from atoms and molecules, it is obvious that the study of single crystals is of extreme importance for a proper understanding of the behavior of polycrystalline materials. As is well known, a great many investigations have been carried out in this field by M. Polanyi and his associates as well as by others. The problem of special interest in the present connection is that regarding the behavior of single crystals under tensile stresses.[1]

The most remarkable phenomenon observed in the case of single crystals is their extremely high ductility. The elongation occurs by slipping along certain planes, which are characteristic for each given type of lattice inasmuch as they are usually those planes in which the density of atoms is greatest or nearly so. The observations on zinc have been summarized by Jeffries and Archer[2] in the following remarks:

> Mark, Polanyi and Schmid in Germany studied the deformation of single crystals by means of X-ray crystal analysis. They found change of orientation at slip planes produced during the ordinary tensile test in single crystals of zinc. Conditions for maximum elongation of single crystals of zinc were determined. Zinc crystallizes with a hexagonal space lattice. The plane of easiest slip is the base of the unit hexagonal prism. When this plane makes an angle of approximately 45 deg. with the wire axis, the crystals are very ductile. Single crystal wires broken in tension at room temperature have shown as much as 600 per cent elongation; broken at 205° C., elongations up to 1700 per cent have been obtained. Although zinc is not regarded as very ductile, these values for elongation are the highest of any known metal. Unless the plane of easiest slip is at an opportune angle with respect to the wire axis, the crystal is relatively brittle. Single crystal wires of bismuth were tested at 200° C. and showed up to 300 per cent elongation. In this case also the plane of easiest slip must make an angle with the wire axis of about 45 deg. or the crystal will be brittle when broken in tension. During the process of elongation of the zinc single crystals, the round wire changed into a flat ribbon. The width of the ribbon was at first slightly wider than the original diameter of the wire. Slip occurred in a plane about 45 deg. from the wire axis and extended across the whole cross-section of the wire in such a manner that the intersection of each slip plane with the wire surface formed an ellipse. The slight widening of the

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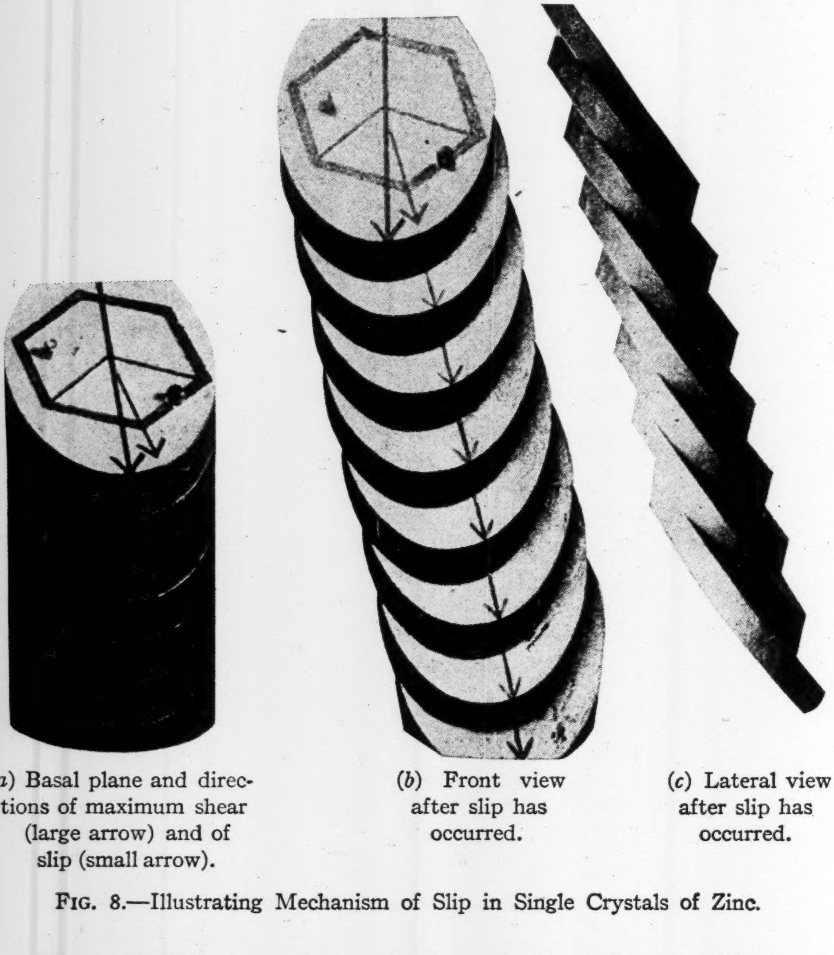
[1] Excellent reviews of these investigations have been published by G. Masing and M. Polanyi, *Ergebnisse der exakten Naturwissenschaften*, Vol. 2, p. 177 (1923), and E. Schmid, *Zeitschrift für Metallkunde*, Vol. 20, p. 370 (1928). See also the papers by H. G. H. Carpenter and C. F. Elam; *Proceedings*, Royal Soc. of London, A, Vol. 100, p. 240 (1921), and *Journal*, Inst. of Metals, Vol. 24, p. 83 (1920).

[2] Z. Jeffries and R. S. Archer, "The Science of Metals," McGraw-Hill Book Co., New York City, 1924.

ribbon was due to the rotation of the elliptical sections. As the elongation increased, the general orientation of the crystal changed, so that the angles of the easiest planes of slip became more nearly parallel with the direction of extension.



(*a*) Basal plane and directions of maximum shear (large arrow) and of slip (small arrow).

(*b*) Front view after slip has occurred.

(*c*) Lateral view after slip has occurred.

FIG. 8.—Illustrating Mechanism of Slip in Single Crystals of Zinc.

Figure 8[1] illustrates the mechanism of slip. Figure 8 (*a*) shows the basal plane, and the large arrow indicates the direction of maximum shear, while the small arrow, which lies in a direction parallel to one of the prism sides, gives the direction of slip. Figures 8 (*b*) and (*c*) show front and lateral views of the arrangement of sections which have slipped on each other.

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[1] From the review by Masing and Polanyi, *Ergebnisse der exakten Naturwissenschaften*, Vol. 2, p. 177 (1923).

The elongation is accompanied by a continued strengthening of the crystal and Polanyi believes that this is due to a bending of slip planes, so that the resistance to slip is increased. The exact mechanism of this strengthening is rather uncertain, but Polanyi assumes that the crystal units themselves undergo inner changes which lead to this result. As a matter of fact, Geiss and von Liempt[1] have concluded from their investigations on single crystals of tungsten that the atoms themselves are deformed by tension, but the only evidence adduced by them for this conclusion is the change in temperature coefficient of resistance.

Polanyi and his associates[2] have also investigated the rupture strength of single crystals. Theoretically the surface energy, σ, rupture strength, S, and elongation ΔL, are related by the equation:

$$S\Delta L > 2\,\sigma$$

which is analogous to Eq. 14 previously deduced by Griffiths and Smekal.

In the case of single crystals of zinc, if we substitute the observed values of S and σ, it is found that the calculated value for ΔL is about 5×10^{-6} cm., which is about 500 times greater than the distance in which the attractive forces should practically vanish.

Accordingly Polanyi concludes:

> The only way out of this difficulty is to assume that the rupture strength of crystals *increases* when the sample used becomes so short that the extension required for rupture becomes less than 5×10^{-6} cm. As L is decreased, S must increase at least inversely as ΔL, and should then attain a value 500 fold greater (about 250 kg. per sq. mm. for NaCl and 1000 kg. per sq. mm. for zinc) when ΔL is equal to the distance in which the molecular attractive forces actually extend, that is, 1×10^{-8} cm. . . .
>
> This increase in rupture strength with decrease in length, L, of the sample should occur both for rock salt and zinc at a length of about 0.5 mm. From this downwards the strength should vary inversely as $1/\sqrt{L}$ until it reaches its highest value at $L = 10^{-7}$ cm. approximately.
>
> This argument shows that the rupture strength of a crystal is not a *definite magnitude*, and therefore cannot correspond to the cohesive forces. . . . From this standpoint, it is to be expected that the rupture strength will be increased by working. The change of the material into a more fine grained state ought to raise the rupture strength, if we make the plausible assumption that the strength of a grain increases with decrease in diameter just like that of a free crystal.
>
> Outside of certain experiments with rock salt, it was especially the determinations of strength of zinc crystals which confirmed the validity of the theory. These measurements were made possible because of the fact that at the tempera-

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[1] W. Geiss and J. A. M. von Liempt, "Significance of Cold Working on the Basis of Electrical Measurements," *Zeitschrift für anorganische Chemie*, Vol. 133, p. 107 (1924); also Vol. 143, p. 259 (1925).

[2] See footnote 1, p. 37.

ture of liquid air zinc crystals are completely non-plastic and therefore rupture readily along one of the crystal planes without any reduction whatever in area. The result was a very low value of the rupture strength of scarcely 3 kg. In these experiments, the same value was obtained independently of the possible orientation of the basal plane with respect to the axis of the wire. Against this observation stands the fact that the rupture strength of polycrystalline zinc (determined on the final cross-section of the sample) is over 60 kg. per sq. mm. or about twenty fold.

Thus the individual grains of a polycrystalline material show the phenomenon of strengthening (by cold-work).

A systematic series of measurements carried out by the author at the temperature of liquid air has extended this discovery still further. The results are summarized in Table VI and *show the increase in rupture strength with fineness of grain and a further increase under the action of cold-working.*

TABLE VI.—RUPTURE STRENGTH OF PURE ZINC AT LIQUID AIR TEMPERATURE, KG. PER SQ. MM.

SINGLE CRYSTAL	COARSELY GRAINED MATERIAL (RECRYSTALLIZED) GRAIN DIAMETER, L, ABOUT 0.2 MM.	FINE GRAINED (RECRYSTALLIZED) GRAIN DIAMETER, L, ABOUT 0.01 MM.	FRESHLY ROLLED
2.5 to 3.5	3 to 6	13 to 18	18 to 40

Assuming the validity of the relation that the rupture strength, S, varies inversely as $1/\sqrt{L}$ for L less than 0.5 mm., and that for $L = 0.5$ mm., $S = 3$ kg per sq. mm., Polanyi finds the following results:

$L = 0.2$ mm., $S = 5$ kg. per sq. mm.
$L = 0.01$ mm., $S = 21$ kg. per sq. mm.

As is evident from Table VI, these calculated values are in agreement with the observed values.

These results may obviously be regarded as a further application of the views suggested by Griffiths and Smekal. As a rather analogous phenomenon Polanyi mentions the observation made by von Hevesy that a very finely crystalline mass of $NaNO_3$ or $CaCO_3$ conducts electricity fifty times better than single crystals of the same material.

The structure produced in metals by rolling and drawing has been investigated not only by Polanyi, Schmid and their associates, but also in the case of tungsten by the staff of the Research Laboratory of the General Electric Co., Ltd., in Wembley, England. The phenomena observed may be summarized by the statement that in all cases the ultimate crystal unit is not destroyed, but merely reoriented so that certain planes lie along the direction of the axis of the wire or in the plane of rolling. In this manner there is produced the well-known fibrous structure of drawn wires.

Undoubtedly further investigation of the properties of single crystals is needed in order to obtain a more intimate knowledge of the cohesive forces between atoms. Such information will furnish a stepping stone in the development of a real atomic theory of the strength of materials.

EFFECT OF TEMPERATURE ON RUPTURE STRENGTH

Further light on the nature of the cohesive forces in metals is obtained by an examination of the numerous observations which have been made on the effect of temperature on tensile strength and other properties of metals. While these observations are, as stated, quite numerous, they have been obtained mostly under conditions which have not been at all uniformly the same over a large temperature range. Not only do we meet with the effects of rapid and slow loading, but also there occur effects due to hardening because of the deformation or changes in grain size. In addition, many metals show one or more allotropic changes of state over a large range of temperatures, and to make the situation still more complex, the engineer has found it convenient to measure ultimate strength on the original cross-section, whereas the physicist requires the true strength as calculated at the breaking point.

However, certain general conclusions regarding the effect of temperature on the strength of materials have been deduced by the metallurgists, and it is, therefore, of interest to refer to these before discussing the observations from the point of view of atomic physics. Jeffries and Archer[1] have postulated that cohesion in metals depends upon the relative magnitudes of the cohesion in the amorphous material between the grains (inter-crystalline cohesion) and that within the grains themselves (intra-crystalline cohesion). The effect of temperature on each of these is shown in Fig. 9. At low temperatures, intra-crystalline cohesion is lower and consequently rupture occurs mostly across the grains, while above a certain temperature (the equi-cohesive temperature), the inter-crystalline cohesion is lower and the rupture occurs generally along grain boundaries. The investigations on the deformation of single crystals discussed in the previous section show that under tension, slipping occurs of adjacent planes and that there are certain planes along which the slip occurs most readily. The same sort of phenomena occur when metals are drawn into wire or rolled into sheets.

The important facts are the existence of a so-called equi-cohesive temperature or inflection temperature and the change in nature of fractures which occur above and below this temperature. As a matter of fact, a study of typical series of observations on the effect of temperature on tensile strength shows a plot which resembles what I will designate as the ideal stress-temperature curve illustrated in Fig. 10 (*a*). Below a certain temperature, which is to be identified with Jeffries and Archer's equi-cohesive temperature, the tensile strength shows a *linear decrease with temperature;* above this temperature the tensile strength decreases *very rapidly* with temperature, and, as will be shown below, is best represented by an *exponential curve.* D. H. Ingall[1] in 1923 drew attention to the existence of two such portions in the stress-temperature plot, and while he described the low temperature part by a linear equation between strength and temperature, he suggested for the higher temperature range an empirical relation of the form $S^n = k/t$ where S is the tensile strength, k and n are constants for any given specimen, and t is the temperature in degrees Centigrade. Actually an exponential relation for the latter range may be made to fit the observations with very satisfactory agreement, as will be shown in a subsequent section.

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[1] Z. Jeffries and R. S. Archer, "The Science of Metals," McGraw-Hill Book Co., New York City, 1924.

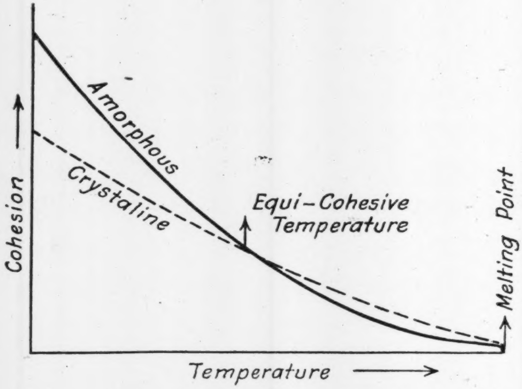

FIG. 9.—Diagram Showing Effect of Temperature on Amorphous Material Between Grains and on the Crystals Themselves as Postulated by Jeffries and Archer.

If we assume the theory of Griffiths as applicable to metals as well as glass, an explanation is obtained of the linear part of the stress-temperature curve. Assuming that the magnitude of the cracks or crevices is independent of temperature, it follows from the relations mentioned already between rupture strength and surface energy that temperature variation in the former will correspond to that in the latter. Now the surface energy decreases approximately linearly with increase in temperature. Hence it follows that the technical strength should follow a similar relation.

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[1] D. H. Ingall, "Experiments with Some Copper Wire: Cohesion a Function of Both Temperature and Cold Work," *Journal*, Inst. of Metals, Vol. 30, p. 171 (1923); also D. H. Ingall, "The Relationship Between Tensile Strength, Temperature and Cold Work in Some Pure Metals and Single Solid Solutions," *Journal*, Inst. of Metals, Vol. 32, p. 41, No. 2 (1924).

Furthermore, since the surface energy increases with decrease in grain size, it follows that the technical strength must also change correspondingly. This is well known to metallurgists. The following observations obtained by Sykes[1] in a study of molybdenum wires may be given in illustration:

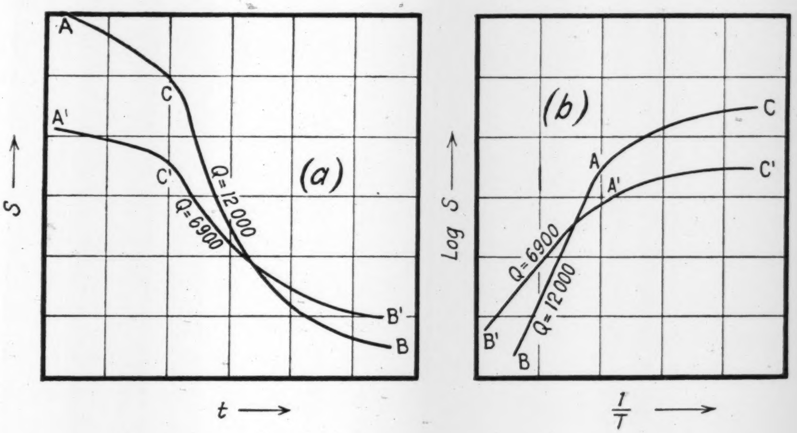

FIG. 10.

(a) Ideal stress-temperature relations, showing linear portion AC (or $A'C'$) at lower temperatures and exponential portion, CB (or $C'B'$) at higher temperatures.

(b) Same data as in (a), but log S has been plotted *versus* $\frac{1}{T}$, whereby the higher temperature portion becomes linear (BA or $B'A'$).

AVERAGE GRAIN DIAMETER, IN.	TENSILE STRENGTH, LB. PER SQ. IN.
Single grain	32 000
0.0011	51 000
0.0002	96 000

However, even if we do not admit the applicability of Griffiths' theory for the case of rupture in metals, there is certainly no reason for expecting any considerable variation with temperature in the cohesional forces between atoms. If the latent heat of vaporization

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[1] W. P. Sykes, "Effect of Temperature, Grain Size and Rate of Loading on the Mechanical Properties of Metals," *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 60, p. 780 (1919).

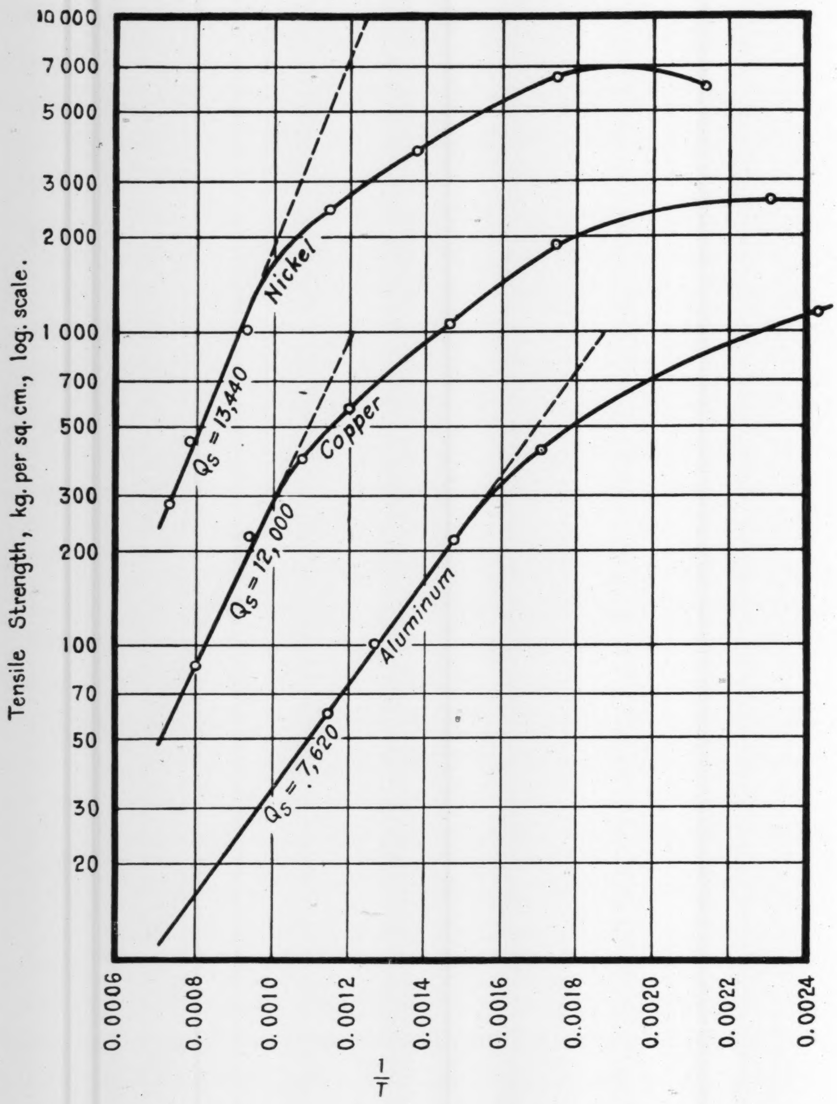

FIG. 11.—Tensile Strength for Copper, Nickel, and Aluminum Plotted on Logarithmic Scale *Versus* $\frac{1}{T}$. (Data from Ludwik.)

is considered as a measure of the lattice energy, it follows from the fact that the latent heat decreases practically linearly with temperature, that the theoretical cohesive strength should also decrease linearly.

There is one factor, however, which is of practical importance in causing deviations from the expected straight-line relation between tensile strength and temperature and that is rate of loading as it affects both brittle and ductile materials. As is well known, a brittle material, like rock salt or glass at ordinary temperatures, has a higher rupture strength for loads applied rapidly than for loads applied slowly. Furthermore, rupture occurs in brittle materials with little or no deformation. On the other hand, in the case of ductile materials where, in general, plastic deformation of the grains takes place as a preliminary stage, there also occurs a hardening because of the cold work and consequently the rupture strength is increased.

Above the equi-cohesive temperature, the rate of decrease in tensile strength with temperature changes quite abruptly, being quite large at first and then becoming less as the melting point is approached. It was found that for a number of metals, observations in this range of temperature give approximately straight lines when log S is plotted against $1/T$, where S is the *tensile strength* and T the absolute temperature in degrees Kelvin.

Figures 10 (*a*) and (*b*) show the same data plotted first on ordinary cross-section paper, and second on semi-logarithmic paper with log S against $1/T$. The points ABC, $A'B'C'$ are indicated on the two sets of curves. The significance of the values of Q_S indicated on the plots is discussed in a subsequent section.

Figure 11 shows graphs plotted in this manner for copper, nickel and aluminum from data obtained in 1915 by P. Ludwik.[1] The observed values are indicated on the graphs. In this case the values of S' used are those given as the corrected tensile strengths in kilograms per square centimeter. Table VII gives the actual data from which the graphs were plotted.

It will be observed that the plots on semi-logarithmic paper show the two distinct portions; that corresponding to higher temperatures (small values of $1/T$) is now linear, or nearly so, with quite a steep slope, while the portion corresponding to lower temperature is a curve which has a much smaller slope throughout its range.

Now the reason for plotting the data in this manner is that in the case of a large number of physical and chemical reactions a similar relation has been found to represent the actual data with a very high degree of agreement. Two well-known examples may be given. The equilibrium constant, K, of a chemical reaction, for example, the reaction $2H_2 + O_2 \rightarrow 2H_2O$, may be represented by an equation of the type:

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[1] P. Ludwik, *Zeitschrift*, Verein Deutscher Ingenieure, Vol. 59, p. 657 (1915), and *Zeitschrift für Physikalische Chemie*, Vol. 91, p. 232 (1916); also given by G. Sachs, "Mechanische Technologie der Metalle," p. 201.

TABLE VII.—DATA ON CORRECTED TENSILE STRENGTH AT VARIOUS TEMPERATURES. (LUDWICK, 1916.)

	TEMPERATURE, T, DEG. ABS.	TENSILE STRENGTH, S', KG. PER SQ. CM.
Aluminum	293	1430
	328	1310
	408	1130
	583	425
	676	215
	783	100
	873	60
Copper	293	3350
	433	2660
	573	1870
	683	1050
	828	565
	923	390
	1066	220
	1243	85
Nickel	293	6650
	468	6020
	573	6460
	728	3770
	866	2440
	1073	1030
	1273	450
	1373	280

$$\log K = -\frac{A}{T} + B \quad \text{(15)}$$

where A and B are constants. Also the vapor pressures of many substances obey a relation of similar form where P, the vapor pressure, is to be substituted for K in the above relation.

In these cases the constant A has a very important physical significance. If natural logarithms are used, then $A = Q/R$ where $R = 1.986$ calories, and Q is the heat of reaction per gram molecule or gram atom at $T = 0$, or the heat of evaporation per gram atom or gram molecule respectively.

Thus the fact that the tensile strength as a function of the temperature apparently follows at higher temperatures a law similar to that observed for a large number of chemical and physical reactions indicates that at these temperatures the change in S with T is due to some physical reaction and furthermore that from the slope of log S against $1/T$ it should be possible to calculate a quantity Q_S which is a measure of the energy of this reaction per gram atom. The second column in Table VIII gives values of Q_S calculated for different metals from data such as those shown in Table VII (obtained by Ludwik), by means of the relation:

TABLE VIII.—HEAT OF REACTION, CALCULATED FROM TEMPERATURE VARIATION OF TENSILE STRENGTH, COMPARED WITH LATENT HEATS OF EVAPORATION AND FUSION.[1]

METAL	Q_S, CALORIES PER GRAM ATOM	APPROXIMATE INFLECTION TEMPERATURE, DEG. K	MELTING TEMPERATURE, DEG. K	LATENT HEAT OF EVAPORATION, CALORIES PER GRAM ATOM	LATENT HEAT OF FUSION, CALORIES PER GRAM ATOM
Nickel..........	13 800	1000	1725	89 440	4278
Tin.............	11 000	425	505	75 000	1593
Copper..........	10 000	800	1356	82 060	2053
Magnesium......	9 500	600	924	41 260	1744
Aluminum.......	6 800	550	933	68 500	2346
Cadmium........	5 000	400	594	26 770	1245
Lead...........	4 350	420	600	48 000	1161

$$\frac{\Delta \log_{10} S}{\Delta (1/T)} = \frac{Q}{2.303R} = \frac{Q}{4.571} \qquad (16)$$

where Δ denotes that the differences in values of $\log_{10} S$ and of corresponding values of $1/T$ respectively are to be taken.

The third and fourth columns in Table VIII give the values of the inflection temperatures as determined from the graphs, and also the melting points, while the last two columns give for comparison the heats of evaporation and of fusion in calories per gram atom. It will be observed that the values of Q_S for any metal *are greater than the heat of fusion and considerably less than the heat of evaporation.*

From data obtained by other investigators, values of Q_S have been calculated which are of the same order of magnitude as those given in the above table. That is, the values lie between 5000 and 15,000 calories per gram atom. In the case of tungsten and molyb-

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[1] Values of L_0 for nickel, copper and cadmium are taken from Table I. The value of L_0 for magnesium and latent heats of fusion are taken from International Critical Tables, Vol. 2, p. 458, Tables 1 and 2. The value of L_0 for lead is estimated from data published by C. Zwikker, *Physica*, Vol. 8, p. 241 (1928).

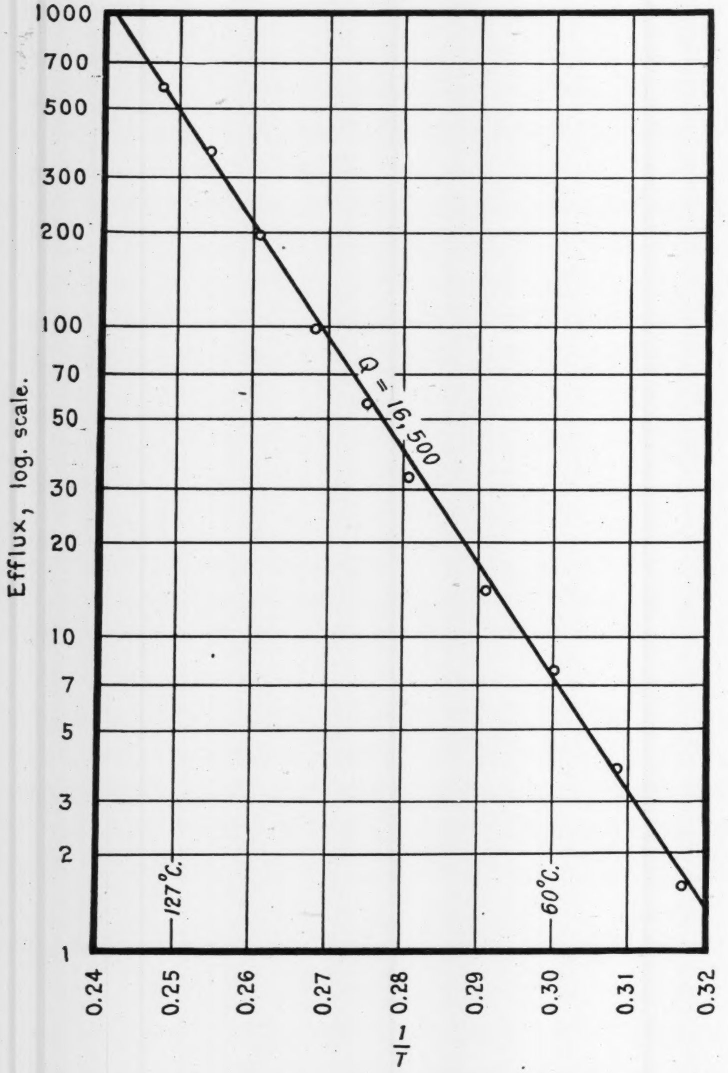

FIG. 12.—Relative Efflux of Tin Through a Small Orifice Plotted on Logarithmic Scale *versus* $\frac{1}{T}$.

denum some results obtained by the writer on the tensile strength of incandescent filaments lead to the conclusion that the values of Q_S for these metals are considerably higher. The conclusion to be drawn, however, from all these data is that the type of reaction of which Q_S is a measure is more nearly like that which occurs in fusion of metals, and that therefore Q_S must correspond to the energy required to cause a translational motion of atoms or some kind of plastic flow.

Observations which have been made on the plastic rate of flow of metals support this hypothesis. Werigen, Lewkojeff and Tamman[1] measured the rate of flow through a small orifice of various metals over a range of temperatures. Table IX gives results obtained with metallic lead. A plot of the logarithm of efflux *versus* $1/T$ gives a very good straight line from which a value of $Q = 13{,}350$ calories per gram atom is obtained. For other metals a similar rapid increase in rate of flow was observed with increase in temperature, and the values of Q calculated from these data are of the same order of magnitude. Figure 12 shows a plot of the data for tin.

TABLE IX.—RELATIVE EFFLUX OF LEAD THROUGH A SMALL ORIFICE.

TEMPERATURE, DEG. CENT.	EFFLUX	TEMPERATURE, DEG. CENT.	EFFLUX
0.5	0.8	60.3	42.4
10.4	1.2	70.0	84.3
20.5	2.3	79.3	157.5
30.4	4.7	89.6	211.5
50.7	22.9		

These observations have suggested to the writer that the physical reaction involved in the measurement of the tensile strength of metals at higher temperatures is essentially one of intermolecular flow or intergranular flow in some cases, and that Q_S is a measure of the energy involved. The importance of the internal viscosity or its reciprocal, fluidity, in accounting for the behavior of metals above the equi-cohesive temperature has been pointed out by Jeffries and Archer in the following remarks:[2]

> Fluidity is the important characteristic of amorphous materials. Plastic deformation takes place by the same kind of flow as in ordinary liquids, except that at low temperatures the viscosity is very great. Whereas the strength of a crystal depends on temperature and is practically unaffected by the duration of loading, the resistance to deformation of an amorphous material not only varies rapidly with the temperature for a given rate or duration of loading,

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[1] N. Werigen, J. Lewkojeff and G. Tamman, "The Rate of Efflux of Some Metals," *Annalen der Physik*, Vol. 10, p. 647 (1903); see also E. C. Bingham, "Fluidity and Plasticity," p. 236.

[2] Z. Jeffries and R. S. Archer, "The Science of Metals," McGraw-Hill Book Co., New York City, 1924.

but if the temperature is constant depends entirely on the rate and duration of loading.

It should be observed in this connection that the coefficients of viscosity of glass and similar materials decrease rapidly with increase in temperature, and it is found that the variation in viscosity with temperature is expressed by a law of the form already mentioned, that is,

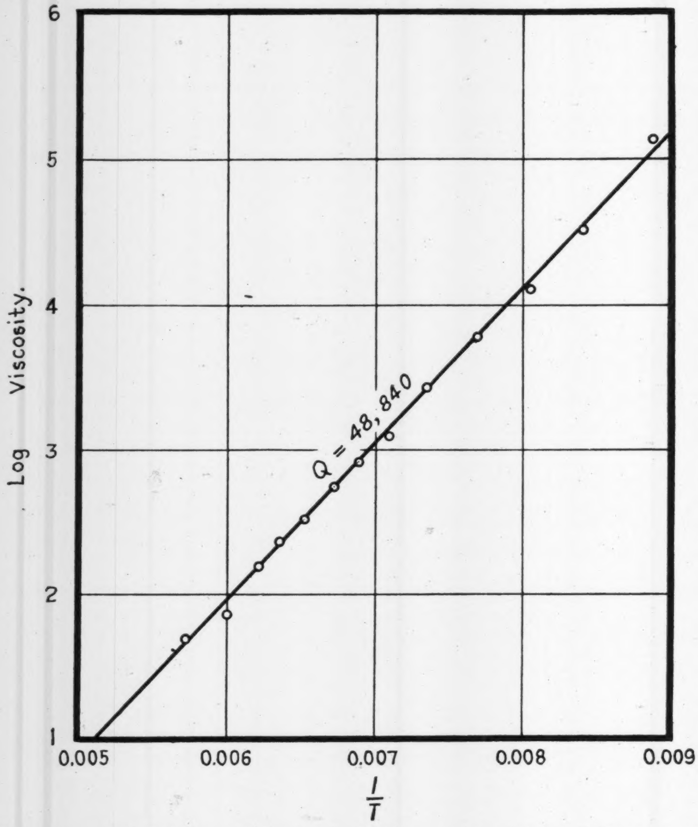

FIG. 13.—Coefficient of Viscosity of Glass as Function of Reciprocal of Absolute Temperature Plotted on Semi-logarithmic Scale.

$$\log n = A + \frac{B}{T} \qquad (17)$$

where n denotes the viscosity and A and B are constants.

Figure 13 shows a plot of log n *versus* $1/T$ of data obtained by E. W. Washburn, G. R. Shelton and E. E. Libman[1] in which the coefficient of viscosity varied about 10,000 fold over the whole range of temperature. From the slope of the straight line, the value $Q = 48{,}000$ calories per gram molecule is calculated.

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[1] E. W. Washburn, G. R. Shelton, and E. E. Libman, "The Viscosities and Surface Tensions of the Soda-Lime-Silica Glasses at High Temperatures," University of Illinois *Bulletin*, Vol. 21, No. 33 (1924).

Now such a large value of Q must indicate a reaction in which fairly strong cohesive forces are involved, and it is interesting to observe that the temperature variation in electrical conductivity of glass also follows a similar law with values of Q which are of the same order of magnitude. In the latter case, the energy involved is that due to ionic dissociation and increased mobility of the ions.

Now the coefficient of viscosity of a fluid is defined as the tangential force per unit area required to cause two surfaces to move with respect to each other at unit velocity. Hence, if we account for the change in tensile strength of a metal at higher temperatures as due to change in viscosity, it is necessary to assume that in taking observations on tensile strength the rate of deformation is kept constant at each temperature. As a matter of fact, as pointed out by Jeffries and Archer, the tensile strength depends upon rate of loading, but if those observations are compared for which the rate of loading is fairly constant, it is found that they plot as straight lines on semi-logarithmic paper if $1/T$ is taken as the axis of abscissa. Furthermore, series of data for different rates of loading are each found to lie on a different line when plotted in this manner.

Ludwik's data, previously mentioned, were obtained under fairly constant rates of loading, and, as stated already, the corrected tensile strengths at rupture were observed. On the other hand, most observers have not worked under uniform conditions of loading over a range of temperatures, nor are the corrected rupture strengths available. However, it is of interest to discuss briefly some of the data which have been obtained by various investigators.

The results of D. H. Ingall[1] have already been referred to. He observed the rapid decrease in tensile strength with temperature and, as mentioned previously, suggested an empirical relation to represent the data. However, his observations are just as well represented above the critical point by the exponential equation of the form $S = S_0\epsilon^{\frac{Q}{RT}}$, while below the equi-cohesive temperature, the values of S are given by the linear relation $S = a - bt$, where a = rupture strength at 0° C., and b is a constant which gives the rate of decrease of S with temperature. Table X gives values of a, b, and Q for copper wires after different degrees of cold working. The inflection point was found to be about 350° C.

---

[1] D. H. Ingall, "Experiments with Some Copper Wire: Cohesion a Function of Both Temperature and Cold Work," *Journal*, Inst. of Metals, Vol. 30, p. 171 (1923).

Ingall interprets the constant *a* as a relative measure of the susceptibility of the material to hardening or strengthening by cold work, and the constant *b* as the susceptibility to softening or annealing with rise of temperature. The percentage increase in *b* is approximately the same as that of reduction by drawing, and hence Ingall concludes that up to 350° C. "the cohesion or tensile strength of the copper used is a linear function of both temperature and cold work." He furthermore remarks as follows:

TABLE X.—CONSTANTS FOR TENSILE STRENGTH OF COPPER AS FUNCTION OF TEMPERATURE. (INGALL.)

SERIES	PERCENTAGE REDUCTION BY DRAWING	TENSILE STRENGTH AT 0° C., *a*, LB. PER SQ. IN.	*b*	*Q*, CALORIES PER GRAM-ATOM
A......................	nil	37 000	43.3	4360
B......................	25	49 000	55.0	5540
C......................	42	56 000	59.0	6890
D......................	50	58 000	64.7	8560
E......................	75	64 000	74.0	7300

TABLE XI.—CONSTANTS FOR TENSILE STRENGTH OF SILVER, ALUMINUM, AND NICKEL AS FUNCTION OF TEMPERATURE. (INGALL.)

METAL	PERCENTAGE REDUCTION	TENSILE STRENGTH AT 0° C., LB. PER SQ. IN.	INFLECTION TEMPERATURE, DEG. CENT.	*Q*, CALORIES PER GRAM-ATOM
Silver..................	nil	28 200	310	3900
	24.4	38 000	260	3620
	49.7	46 800	220	3200
	74.7	53 400	180	3270
Aluminum..............	nil	14 600	165	3600
	25.2	18 300	225	4700
	49.3	21 800	210	6350
	75.3	24 000	175	4500
	65.9	23 400	185	4370
Nickel..................	nil	68 500	500	3230
	27.9	99 000	500	2700
	50.9	117 500	515	3460
	75.3	130 000	525	3665

> The change in the behavior of the material at the higher temperatures was at once evident from an ordinary eye examination of the fractures.

> Up to the critical inflection temperature of 350° C. the material fractured in the ordinary way of a ductile metal with considerable local necking and the usual cup and cone break.
>
> Beyond 350° C. the local necking and the cup and cone quickly disappeared and the fracture had the appearance of that of an extremely brittle material.

In a subsequent paper,[1] Ingall has also investigated in a similar manner the pure metals silver, aluminum, nickel and two solid solutions, a 70-30 brass and 80-20 cupro-nickel. The plots of the data show the linear portion below a certain inflection temperature and a very rapid decrease in tensile strength above this temperature. Table XI gives values of the inflection temperature and of Q for the different metals after various conditions of treatment.

In these cases the values of Q vary irregularly with the percentage reduction, and it is of interest to note that apparently Q has the highest value for the metal of lowest melting point. These observations suggest that there are other factors influencing the value of Q, of which we are at present ignorant. The following remarks by Ingall on the interpretation of the higher temperature data are significant:

> In a recent paper the author summarized the details from a bibliography of elevated temperature mechanical tests on metals, and showed that the results obtained were compatible with the view that metals at elevated temperatures behaved as if consisting of a partly crystalline and partly viscous state. It is suggested that the curved line or higher temperature curve denotes the condition of the material where *viscous flow may take place*, and that the critical inflection temperature marks the minimum limiting temperature for this phenomenon. *For this reason it is suggested that the straight line or lower temperature curve is the all important one to the engineer for, at a temperature beyond it, that is, beyond the critical inflection temperature which limits it, a metal or alloy will only be of mechanical use when it is subjected to a stress which is insufficient to overcome the resistance to flow due to friction, etc., or a cycle of stresses which do not give time for flow.*
>
> On the assumption of the possibility of viscous flow at temperatures above the critical inflection temperature, the following suggested explanation is put forward.
>
> 1. Below the critical inflection temperature.
>
> The possibility of general deformation, that is, of elongation or general reduction of cross-sectional area in a tensile test, is dependent upon a slipping grain being able to hand over its excess stress to its neighbors. (*a*) The extent to which this may be done will be a measure of the elongation (or general reduction); when the limit of this action is reached any increase of the general stress will produce a local breakdown at the weakest spot, and (*b*) the extent of this breakdown, which measures the local contraction, will depend on the extent to which the local grains concerned can further slip.

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[1] D. H. Ingall, "The Relationship Between Tensile Strength, Temperature and Cold Work in Some Pure Metals and Single Solid Solutions," *Journal*, Inst. of Metals, Vol. 32, p. 41, No. 2 (1924).

2. Above the critical inflection temperature.

The conditions pertaining to (1) will hold, but the following factors will play an important part.

(*a*) The presence of viscous material capable of flow.

(*b*) The relative amounts of crystalline and viscous material.

(*c*) The size of the crystalline material, for the larger the lumps the greater obstruction will they offer to flow.

(*d*) The amount and the *nature* of the impurities in the crystalline masses, which will affect the extent to which they can aid viscous flow by their own slipping.

(*e*) Temperature. The higher the temperature the easier will viscous flow be able to take place; this affects (*a*), (*b*) and (*c*) above. Rosenhain has demonstrated the effect of temperature on space lattice distortion and resistance to slip, which affects (*d*) above.

(*f*) Time for flow.

The effect of temperature on the tensile strength of nickel has been investigated by Sykes.[1] Above 600 to 700° C. a rapid decrease in strength was observed. From his data, which give fairly straight lines on semi-logarithmic paper, the values of Q obtained range from 6900 calories for annealed wires to 12,500 calories for wires drawn cold. For a copper-aluminum alloy the values of Q, from data obtained by the same investigator, vary from 5500 to 10,000 calories, the value increasing with amount of cold work.

TABLE XII.—VALUES OF Q FOR COPPER WIRE AFTER DIFFERENT TREATMENTS.

REFERENCE IN JEFFRIES' PAPER	TREATMENT OF WIRE	Q_S
Table 1	Drawn cold from 0.125 to 0.025 in.	7500
Table 2	Drawn cold as above, then annealed at 750° C.	5500
Table 3	Drawn cold from 0.042 to 0.025 in.	6200
Table 4	Drawn cold from 0.031 to 0.025 in.	6200
Table 5	Drawn at 150° C. from 0.075 to 0.025 in.	6200

The same conclusion is to be drawn from results obtained by Z. Jeffries[2] on copper, as is shown in Table XII which gives values of Q_S for different wires.

It will be observed that in all the cases mentioned, the value of Q is greater for cold-drawn than for annealed wires; furthermore, that the greater the reduction in drawing, the higher the value of Q. Also from the published data it would appear that in general the value of Q is affected only slightly by variation in rate of loading. However, it is not possible to draw any definite conclusion on this point from the available observations.

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[1] See footnote 1, p. 43.

[2] See footnote 3, p. 33.

As is well known, the grain structure of a cold-drawn wire is much finer than that of annealed wire and it therefore follows from the above observations that fine-grained structure leads to increase in Q; that is, the energy required in overcoming resistance to flow is greater for the fine-grained material. Because of this higher value of Q, the tensile strength of the latter drops below that of the coarser material at higher temperatures; although at lower temperatures the order of strengths is reversed. It is also the existence of this reversal at higher temperatures that leads Ingall to the conclusion that "the larger the lumps the greater obstruction they will offer to flow."

Jeffries has discussed in the paper referred to previously the deformation of metals as affected by grain size and temperature from the point of view of the amorphous metal hypothesis. In terms of this theory, the value of Q is a measure of the energy of deformation of the amorphous material and it is therefore of interest to quote some of his remarks on the effect of temperature on the strength of this material.

After describing certain tests on the hardness of coarse-grained and fine-grained tin and lead, he proceeds as follows:

> This viscosity is typical of amorphous materials. The amorphous phase deforms slowly because the deformation has to be transmitted from atom to atom. Any change in shape, in general, will involve the change of position of every atom relative to every other atom in the deformed region. This is not true in the deformation of crystalline material. Blocks of the crystalline material consisting of millions of atoms move by displacement along cleavage or gliding planes and the atoms within one such block or fragment remain at the end of deformation in about the same positions, relative to one another, as they had before deformation. The atoms within one fragment, however, have changed their positions with reference to atoms of another fragment. The movement of these relatively large blocks of atoms would take place much more rapidly than a movement in which each atom had to move relative to each other atom.

In this connection, Jeffries has also emphasized the point that "the cohesion of amorphous materials depends on the time of application of the load that measures it." A large load for a short time and a relatively smaller load for a longer time are equivalent. This is characteristic of viscous materials, and consequently is in agreement with the theory that Q is a measure of the temperature coefficient of viscosity of the inter-crystalline material.

TEMPERATURE COEFFICIENT OF DIFFUSION AND GRAIN GROWTH

There are other phenomena characteristic of metals, glass and other materials at higher temperatures which are in accord with the above interpretation of the temperature variation in tensile strength.

Some years ago, I. Langmuir and the writer[1] found that the rate of diffusion of metals into each other as a function of the temperature could be represented by an exponential relation of the same type as that suggested in the present sections, that is, of the form

$$D = D_0 \epsilon^{-\frac{Q}{RT}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (18)$$

where D = diffusion constant at $T°$ K.

D_0 = constant.

Q = heat of diffusion.

The values of Q obtained for different cases are shown in Table XIII.

The mechanism of diffusion suggested by Langmuir in explanation of these observations was that diffusion occurs by an interchange of place between adjacent atoms.[2] Thus the probability that an atom, say of thorium, will replace an atom of tungsten in the crystal lattice is proportional to $\epsilon^{-\frac{Q}{RT}}$, and Q represents the energy required for a gram-atom of thorium to replace an equivalent amount of tungsten. That is, Q in this case is a measure of the mobility of the thorium atoms through the tungsten lattice. The magnitudes of the heat of diffusion for the various cases mentioned in Table XIII are evidently considerably greater than those derived from tensile strength data. This is in accordance with the view that while in the latter case the reaction involved is either a motion of relatively large units, such as the grains, or a motion of similar atoms past each other, the reaction in diffusion is one in which individual atoms of one kind have to interchange places with atoms of another kind.

TABLE XIII.—HEATS OF DIFFUSION FOR VARIOUS METALS CALCULATED FROM TEMPERATURE-COEFFICIENT OF DIFFUSION.

DIFFUSING METALS	HEAT OF DIFFUSION, Q, CALORIES
Thorium in tungsten	94 000
Carbon in tungsten	80 000
Gold in lead	13 300
Zinc in iron	38 000
Zinc in nickel	29 000
Zinc in copper	27 000

Grain growth is another phenomenon which, as is well known, increases with temperature. The experiments of Bassett and Davis[3] on grain growth in 68–32 brass cold rolled 50.9 per cent offer a good illustration. Here again it is found that the average grain area increases with the temperature according to Eq. 18 where D may be taken to represent the average area of a grain. From the plot shown in Fig. 14, the value $Q = 28{,}400$ is obtained, which is of the same magnitude as the values derived for diffusion phenomena, and much greater than that involved in viscous flow.

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[1] S. Dushman and I. Langmuir, "The Diffusion Coefficient in Solids and Its Temperature Coefficient," *Physical Review*, Vol. 20, No. 1, July, 1922.

[2] This subject has been discussed very fully by Z. Jeffries, "The Trend in the Science of Metals," Am. Inst. Mining and Metallurgical Engrs., May, 1924.

[3] Quoted by Z. Jeffries and R. S. Archer, "The Science of Metals," p. 98.

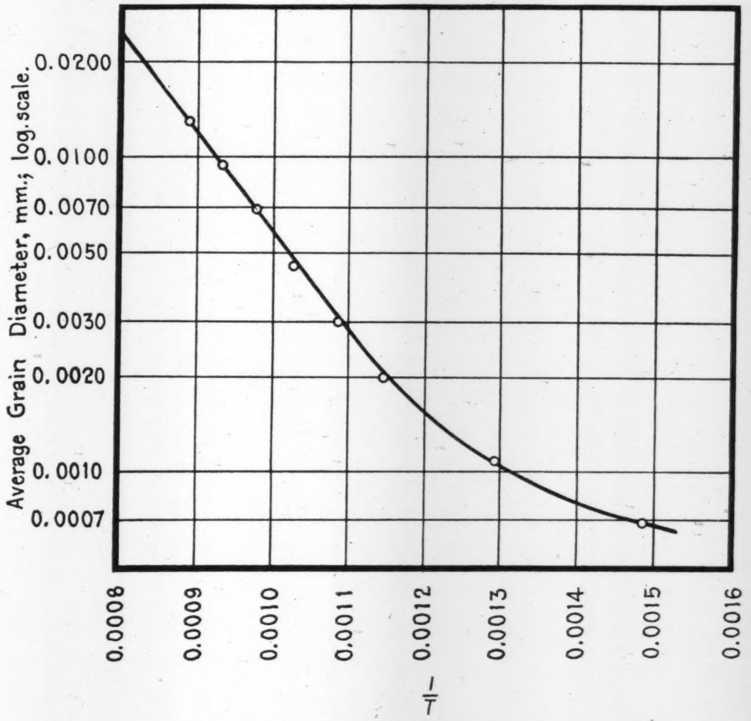

FIG. 14.—Variation in Grain Size with Reciprocal of Absolute Temperature for Brass, Plotted on Semi-logarithmic Scale. (Bassett and Davis.)

The rate of grain growth in tungsten ingots sintered at various temperatures has been investigated by Jeffries[1] and from his data, a plot of which is shown on semi-logarithmic paper in Fig. 15, the value $Q = 80{,}000$ is obtained. This value is so nearly equal to those observed for diffusion of thorium and carbon in tungsten that it would appear logical to conclude that both phenomena depend upon rates of migration of atoms.

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[1] Z. Jeffries, "Metallography of Tungsten," *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 60, p. 588 (1919).

The numerous observations which have been made on recrystallization, by Czochralski and others, furnish additional illustrations of the application of the same point of view.[1] The grain size increases rapidly with temperature at rates which indicate large values of Q. Furthermore, cold working of the metal decreases the rate of grain growth. This corresponds to an increase in Q with working, just as

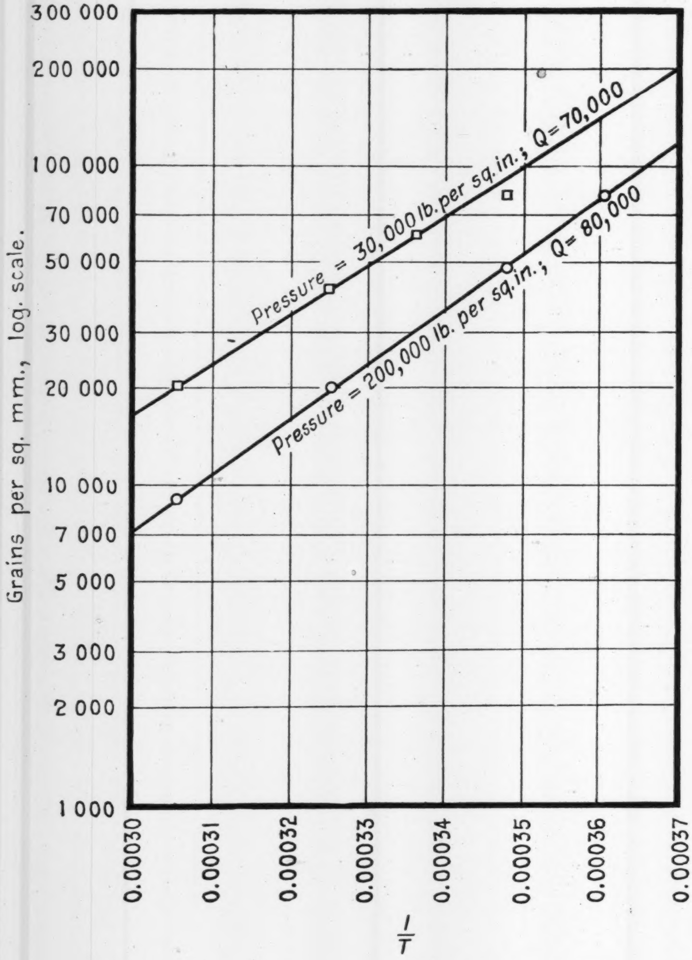

FIG. 15.—Variation in Grain Size with $\frac{1}{T}$ for Tungsten Ingots at two Different Pressures. (Jeffries.)

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[1] For an excellent summary of these investigations see G. Sachs, "Mechanische Technologie der Metalle," pp. 171–173.

was observed for the case of tensile strength tests. That is, the resistance of motion of atoms in forming grains is increased by the mechanical working, or, expressing the same idea in other words, the probability that atoms in adjacent grains will readjust themselves to form a continuous lattice is decreased because these grains have been deformed by working.

APPLICATION OF PROBABILITY CONCEPTS TO STUDY OF COHESION

In the previous discussion, the exponential relation for strength as a function of temperature was introduced as a semi-empirical deduction based on analogy with rates of physical and chemical reactions. There is, however, another method of arriving at the same result which is based on the so-called Boltzmann conception of the probability of occurrence of a physical phenomenon.

Let us consider again the evaporation of a solid metal, such as copper, at temperature T. Let L_0 denote the latent heat of evaporation. According to Boltzmann the amplitudes of vibration of the different atoms are not equal, but vary from zero to infinity in accordance with a definite distribution law. There are, therefore, at any given temperature a certain number of atoms for which the energy of vibration is in excess of that required to separate them completely from the lattice. The probability of occurrence of atoms with such high energy content will govern the rate of evaporation, and, according to Boltzmann, this will be given by

$$W = W_0 \epsilon^{-\frac{L_0}{RT}} \qquad (19)$$

where W denotes the probability that an atom will evaporate at the temperature T. At $T = \infty$, $W = W_0$, and thus attains its maximum value, while W is decreased as T is decreased.

From this standpoint, the interpretation of the mechanism of rupture at higher temperatures would be somewhat as follows:

The probability that an atom in a grain boundary will be able to move past an adjacent atom is proportional to $\epsilon^{-\frac{Q}{RT}}$, where Q represents the energy involved in this motion per gram atom of metal. Now the tensile strength is a measure of the reciprocal of this probability. Hence the tensile strength will vary with temperature according to the relation stated previously.

The same ideas have also been expressed in a paper by R. Becker[1] which presents a most important contribution to the study of plasticity, hardening, and recrystallization in metals. We have therefore considered it worth while to present a summary of this paper in the present connection.

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[1] R. Becker, "Plasticity, Hardening and Recrystallization in Metals," *Zeitschrift für techn. Physik*, Vol. 7, p. 547 (1926).

"The mechanism of plasticity of single crystals has been investigated thoroughly on the geometrical crystallographic side. Under the action of shearing stresses there occurs a gliding of the material on definite crystallographic planes, and in some cases also the direction of gliding within the surface is determined crystallographically. The lowest shearing stress required to initiate the gliding is known as the elastic limit. . . . This limit is never measured exactly; it is possible only to state that load which, with the measuring arrangement used, is just sufficient to produce an observable extension." He then discusses Frenkel's theory[1] in which the gliding is assumed to be similar to that of two toothed surfaces. The objection to this theory is that the stress, S, would produce a shearing break and not a plastic deformation. "It is characteristic," he states, "of plastic deformation that it must proceed with quite a *definite velocity*. To account for this I have therefore suggested the following mechanism, which is supported by some older ideas of Tamman:

> "We assume that on the body to be deformed we apply such a load that on the crystallographic gliding plane a shearing stress s is effective which is less than the limiting stress, S, and we consider especially a definite gliding plane. In consequence of the irregular thermal motion, the force acting along the gliding plane will undergo irregular oscillations about the average value, s. If now the limiting value, S, is exceeded for a short time, a gliding action will suddenly be started during such an instant. . . . The plasticity of a crystal is thus affected by thermal oscillation phenomena, which leads to the consequence that at one instant gliding will occur on one plane and at another instant on another plane, the occurrence of gliding on any particular plane being governed by the law of probability."

Becker calculates that the rate of plastic deformation, u, will be given by a relation of the form:

$$u = u_0 \epsilon^{-\frac{a(S-s)^2}{RT}} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots(20)$$

where a is a constant involving the modulus of elasticity, and S and s have the significations assigned above.

Becker gives no actual data, but states that certain observations have shown that the value of u increases about three fold for a 10 degree rise in temperature. The quantity $a(S-s)^2$ represents an amount of energy, Q, and the temperature coefficient stated by

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[1] Frenkel, "On the Theory of Elastic Limit and Strength of Crystalline Bodies," *Zeitschrift für Physik*, Vol. 37, p. 572 (1926).

Becker would lead to values of Q which are of the order of magnitude of 10,000 or higher.

In view of the discussion already presented, the following remarks of Becker are worth quoting: "According to this point of view, it is of interest for the physical investigation of plasticity in metals to determine thoroughly the manner in which the velocity of flow varies with applied force and temperature. In technical investigations hitherto carried out, these questions have played only a minor rôle."

In discussing the theory of hardening by deformation, Becker remarks that while it has not yet been found possible to describe the phenomena in detail from an atomic standpoint, investigation has disclosed changes in physical properties which are significant.

The most important of these changes is the increase in free energy in the hardened metal as compared with the original, as shown by electrochemical potential measurements. von Hevesy has also shown that diffusion of radioactive lead in a single crystal of lead is at least 10,000 times smaller than in rolled lead. A diffusion can, however, occur only through interchange of places among individual atoms owing to thermal agitation and consequent disturbance of the lattice. Such a phenomenon is difficult to understand in an undisturbed lattice and von Hevesy's observation on the failure of diffusion in a single crystal is in agreement with this. Similar observations have been made by Geiss and von Liempt,[1] and independently by the writer, on the diffusion of thorium in tungsten. In this case diffusion seems to occur along grain boundaries and is, therefore, more rapid at any given temperature for fine crystalline wire than coarser structure. The conclusion is therefore drawn by Becker that:

"Lattice disturbances are the necessary condition for the subsequent occurrence of interchange of position among atoms. At the same temperature these interchanges occur the more frequently the further the structure is removed from that of a single crystal."

In recrystallization, the disturbed lattice is restored to the undisturbed state and the elementary mechanism is again an atomic interchange of position. Becker postulates that the probability of occurrence of such an interchange, W, is given by the relation

$$W = W_0 \epsilon^{-\frac{Q}{RT}} \qquad (21)$$

where Q represents the difference in potential energy of the atoms in the deformed lattice as compared with the undeformed. This accounts for the observation that temperature of recrystallization is decreased by cold working. Similar considerations lead to the conclusion, as has already been pointed out, that grain growth must increase with temperature, and Becker has discussed these phenomena in further detail. His point of view thus agrees to a great extent with that expressed by the writer.

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[1] W. Geiss and J. A. von Liempt, *Zeitschrift für Metallkunde*, Vol. 16, p. 317 (1924).

As is well known, a great many investigations have dealt with the subject of recystallization and grain growth from a metallurgical point of view. However, from the point of view of the physicist relatively very little has been accomplished towards attaining a more profound understanding of the atomic processes involved. It is obvious that grain growth is accompanied by migration of atoms from places of higher to those of lower potential energy, and numerous observations on the growth of crystals by deposition from solutions or vapors show that atoms are mobile at temperatures that are very much lower than the melting points of the crystals. Undoubtedly the further investigations of the phenomena of sintering and recrystalization in general are of great importance and Becker's contribution to the theoretical aspects of the subject are therefore extremely significant.

An attempt has also been made by W. Braunbek to apply the Boltzmann theorem to the phenomena of fusion[1] and electrolytic conductivity[2] of ionic compounds. Thus he has derived the exponential relation previously discussed for electrolytic conductivity of crystals and glass as a function of the temperature in terms of the lattice constant. From his results the value for Q is found to be 21,250 calories per gram molecule.

It is therefore evident that this point of view is applicable to a large class of phenomena in which temperature is an important factor, and undoubtedly many more interesting results will be obtained from further systematic investigations on the effect of temperature on the cohesive properties of materials.

CONCLUDING REMARKS

The problem of cohesion in solids is an extremely complex one, and while an ultimate theory of cohesion must be able to interpret all the observed phenomena in terms of atomic structure, it is evident that only a beginning has been made towards solving the problem along such lines. The theories of the solid state and of the effect of temperature on rupture strength as presented in this paper give no mechanism of cohesion, but, like all thermodynamical arguments, represent working conceptions which yield satisfactory results without the necessity of appealing to models. These theories must, therefore, be supplemented in the future by more mechanistic views which shall reveal the observed phenomena as a direct result of electronic configurations and atomic vibrations.

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[1] W. Braunbek, "On the Lattice Dynamics of the Melting Process," *Zeitschrift für Physik*, Vol. 38, p. 549 (1926).

[2] W. Braunbek, "A Lattice Theoretical Calculation of the Electrolytic Conductance of Rock Salt Crystals," *Zeitschrift für Physik*, Vol. 44, p. 684 (1927).

The writer appreciates more than anyone else the obvious incompleteness of the discussion of the problem as presented in this paper. A large number of observations bearing on the subject have either not been mentioned at all, or have been treated very briefly.

Thus recent investigations have shown that small amounts of certain impurities in metals have a considerable effect on the strength of alloys; as an illustration we may cite the effect of 0.07 per cent of nitrogen in iron.[1] A comprehensive atomistic theory of the strength of materials will undoubtedly account for such observations either by assuming that the added molecules arrange themselves in the crystal lattice in such a manner as to prevent plastic deformation along certain planes, or else that these impurities serve to neutralize the effects of accidental cracks and other inhomogeneities in the lattice structure and thus lead to increased strength. For it must be emphasized that in no cases have we been able to exceed technically the molecular cohesive strength. As Jeffries and Archer have remarked: "Instead of inquiring why metals are so strong, it may be asked why metals are so weak."

The very important investigations and speculations on hardening properties of alloys have been omitted, since these are discussed fully in different metallurgical treatises, while there is very little that can be contributed at present to these subjects from the point of view of atomic physics.

Other subjects such as those of grain growth and recrystallization have been discussed rather briefly, although their significance in leading to an understanding of the nature of the ultimate molecular forces is very great.

It is to be hoped that the discussion of the subject in this paper will not only give the reader a summary of our present knowledge, but will also serve to stimulate further investigation along the lines already indicated. For after all, theories and conceptions are but the ephemeral frills of any branch of science, while the observations are the essential aids whereby we approach the truth asymptotically as it were.

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[1] The writer is indebted to Zay Jeffries for drawing his attention to this observation.

In conclusion the writer desires to express his appreciation of the assistance which he has obtained in the preparation of this paper from discussions with Dr. Zay Jeffries in Cleveland, and also with his colleagues in the Research Laboratory at Schenectady, especially Dr. S. L. Hoyt and Mr. T. S. Fuller.

SYMPOSIUM ON PHYSICAL PROPERTIES OF CAST IRON

INTRODUCTION

By H. Bornstein[1]

The term "cast iron," like the term "steel," covers a wide range of materials. In the case of steel, rapid progress has been made during the past two decades in classifying various types of steels and determining their physical properties. As a result, the engineer has become familiar with the large range of steels and is able to select the material which suits his particular purpose.

There has not been the same progress in the classification of cast irons and in the determination of their physical properties. Furthermore, accurate information in respect to cast iron has not been very widely disseminated to the engineering profession. As a consequence, the engineer has frequently looked with suspicion on cast iron as an engineering material.

The number of specifications covering cast iron is small and the information given as to physical properties is not at all comprehensive. Considerable progress has been made in cast-iron metallurgy so that irons with high physical properties are available to industry. But our specifications have not as yet caught up with our practices. Occasionally the proper grade of cast iron is not specified and the results are not satisfactory. Cast iron is a reliable engineering material and knowledge as to its properties is necessary to encourage its proper use in industry.

This matter received considerable discussion at a meeting of Committee A-3 on Cast Iron at Atlantic City last year, and it was suggested that a symposium on the physical properties of cast iorn be held at the 1929 annual meeting. Accordingly, the following committee was organized to act in an advisory capacity to the Committee on Papers in arranging the symposium:

J. W. Bolton
F. B. Coyle
E. J. Lowry
J. T. MacKenzie
R. S. MacPherran
H. Bornstein, chairman

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[1] Director of Laboratories, Deere and Co., Moline, Ill.

The purpose of this committee has been to attempt to assemble a group of papers which would give information on the classification of cast irons and on the important physical properties.

It is realized that only a beginning has been made and that considerably more information is necessary. The presentation is far from complete and it is hoped that considerable information will be brought forth in the discussion of these papers.

CLASSIFICATION OF GRAY-IRON ALLOYS

By J. W. Bolton[1]

Synopsis

It is necessary to classify gray irons before intelligent general study can be undertaken. Five factors influencing the properties of gray iron castings are enumerated. The author advocates a primary classification based upon chemical analysis, and a secondary classification based upon cooling rate. In this latter connection he shows how a mathematical formula can be applied to determine relative cooling rates of simple shapes. Experimental proof of the validity of application of this formula is given.

The term gray cast iron is generic rather than specific. It covers a series of alloys of iron, silicon and carbon, with various percentages of other metals and metalloids. All these alloys in the as-cast condition contain graphitic carbon, associated with pearlite or ferrite, or both. While no recent comprehensive definition has been accepted as universal standard, most commercial gray irons contain between 2.50 and 3.75 per cent of total carbon, and 0.50 to 3.00 per cent of silicon. A very wide range of physical properties is found among the alloys of the series. Tensile strength, for example, may be from 20,000 to 60,000 lb. per sq. in. Brinell hardness varies from 100 to 240. The wide variations in physical properties suggest like variations in service obtainable in the field, and such is the case. Unfortunately the foundry trade has been slow to recognize these variations, or at least to encourage research to properly measure and define them. As a result improper grades often have been furnished, the resultant failures giving an uncertain reputation to this worthy engineering material.

In order to specify intelligently, the engineer must be informed as to what grade best meets his requirements. Yet little such information is at his disposal. To define grades of iron, it is necessary to draw up a classification of the gray-iron alloys. Classifications might be established on various bases. The most logical seems a primary classification according to causative factors. These are:

1. Chemical composition—initial, incidental and final;
2. Structural and mechanical makeup of charge;
3. Melting process;

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[1] Chief Chemist and Metallurgist, The Lunkenheimer Co., Cincinnati, Ohio.

4. Thermal and mechanical history—furnace spout to cooled casting; and
5. Design and workmanship.

Before proceeding to discussion of the many factors involved, it should be mentioned that the classification appears much more complex on paper than it works out in every-day shop practice. Indeed there seems every reason to believe that a few years of coordinated cooperative work will remove much of the mystery and uncertainty now pervading certain portions of the gray-iron field. The present paper is preliminary and devoted largely to classification of causative factors. With assembly of test data, physical and mechanical effects will be rapidly assigned to their proper niches, and a set of worthy engineering data should result.

CHEMICAL COMPOSITION

As noted above, gray cast iron contains carbon and silicon. Commercial irons also contain substantial percentages of phosphorus, manganese and sulfur. In addition to the above metals and metalloids other elements may be present. Nickel and chromium often are present as deliberate alloy additions, whereas small percentages of titanium, copper, etc., are reduced from certain ores; and dissolved and occluded gases are to be found.

Structurally gray iron is an iron-carbon alloy, its principal microscopic components being pearlite (cementite-ferrite eutectoid), ferrite and graphitic carbon. Therefore the percentage of total carbon has a very important bearing on all the physical and mechanical properties of the metal. For this reason we have based the initial classification according to chemical composition on the percentages of total carbon. There are five groups, as shown in Table I.

Within any iron-carbon alloy of the gray-iron series, of given carbon content and like (entire) thermal history, the relative amounts of pearlite, ferrite and graphite can be controlled to a great degree by the percentage of silicon present. In addition to its effect on melting point, position of eutectic and thermal critical points, this element has a very marked influence on the degree of graphitization. Indeed the foundryman learned to utilize this metalloid long before he reached an understanding of the effects of total carbon. Silicon control is very important, and the first sub-classification according to chemical composition is based on the percentage of this element. There are eight groups, listed under each carbon classification (see Table I).

Phosphorus is an important element, a constituent of the component known as steadite. It is well known that phosphorus has some effect on the fluidity of cast iron, and steadite diminishes the deflection under ultimate load and is generally held to lower the impact resistance.

TABLE I.—CLASSIFICATION OF CAST IRONS ACCORDING TO SILICON AND CARBON CONTENT.

Group	Total Carbon, per cent	Silicon, per cent	Carbon + 0.30 per cent Silicon, per cent	Carbon + Silicon, per cent
No. 1	Above 3.50	Above 2.50	4.25 up	6.00 up
No. 2	" "	2.26 - 2.50	4.21 up	5.88 up A
No. 3	" "	2.01 - 2.25	4.14 up A	5.63 up C
No. 4	" "	1.76 - 2.00	4.06 up B	5.38 up E
No. 5	" "	1.51 - 1.75	3.99 up C	5.13 up G
No. 6	" "	1.26 - 1.50	3.91 up D	4.88 up I
No. 7	" "	1.01 - 1.25	3.83 up E	4.63 up K
No. 8	" "	0.76 - 1.00	3.76 up F	4.88 up M
No. 9	3.26 - 3.50	Above 2.50	4.13 up A	5.88 up A
No. 10	" "	2.26 - 2.50	4.09 B	5.76 B
No. 11	" "	2.01 - 2.25	4.02 C	5.51 D
No. 12	" "	1.76 - 2.00	3.94 D	5.26 F
No. 13	" "	1.51 - 1.75	3.87 E	5.01 H
No. 14	" "	1.26 - 1.50	3.79 F	4.76 J
No. 15	" "	1.01 - 1.25	3.71 G	4.51 L
No. 16	" "	0.76 - 1.00	3.64 H	4.26 N
No. 17	3.01 - 3.25	Above 2.50	3.88 up D	5.63 up C
No. 18	" "	2.26 - 2.50	3.84 E	5.51 D
No. 19	" "	2.01 - 2.25	3.77 F	5.26 F
No. 20	" "	1.76 - 2.00	3.69 G	5.01 H
No. 21	" "	1.51 - 1.75	3.62 H	4.76 J
No. 22	" "	1.26 - 1.50	3.54 I	4.51 L
No. 23	" "	1.01 - 1.25	3.46 J	4.26 N
No. 24	" "	0.76 - 1.00	3.39 K	4.01 O
No. 25	2.76 - 3.00	Above 2.50	3.63 up H	5.38 E
No. 26	" "	2.26 - 2.50	3.59 H	5.26 F
No. 27	" "	2.01 - 2.25	3.52 I	5.01 H
No. 28	" "	1.76 - 2.00	3.44 J	4.76 J
No. 29	" "	1.51 - 1.75	3 37 K	4.51 L
No. 30	" "	1.26 - 1.50	3.29 L	4.26 N
No. 31	" "	1.01 - 1.25	3.21 M	4.01 O
No. 32	" "	0.76 - 1.00	3.14 N	3.76 P
No. 33	2.51 - 2.75	Above 2.50	3.38 up K	5.13 G
No. 34	" "	2.26 - 2.50	3.34 K	5.01 H
No. 35	" "	2.01 - 2.25	3.27 L	4.76 J
No. 36	" "	1.76 - 2.00	3.19 M	4.51 L
No. 37	" "	1.51 - 1.75	3.12 N	4.26 N
No. 38	" "	1.26 - 1.50	3.04	4.01 O
No. 39	" "	1.01 - 1.25	2.96	3.76 P
No. 40	" "	0.76 - 1.00	2.89	3.51

NOTE.—Manganese on most foundry irons runs between 0.45 and 0.80 per cent, within which range no subclassification seems necessary. Phosphorus subdivisions might be 0 to 0.15, 0.16 to 0.30, 0.31 to 0.50, 0.51 to 0.70 and 0.71 to 1.00 per cent; this element influences the stiffness of the metal and also is a factor in its fluidity and hardness. At the present time, no logical subdivisions on sulfur content have occurred to the author; perhaps the sulfur-manganese ratio will furnish a basis for further classification. Nickel and chromium have powerful influences on cast iron, and irons containing them exhibit somewhat different characteristics than "straight" silicon-carbon irons

Five phosphorus sub-sub-classifications, say 0 to 0.15, 0.16 to 0.30, 0.31 to 0.50, 0.51 to 0.70, and 0.71 to 1.00 per cent, will probably prove ample.

Manganese and sulfur, in ordinary amounts, appear to have slight influence on the usual mechanical properties of gray iron, hence no classification is needed. It must be remembered, however, that in the irons approaching the chill condition, the absolute and relative amounts of these elements influence the degree of graphitization. Therefore they cannot be considered as wholly inconsequential.

In so far as other elements are concerned, it seems wholly impractical at present to attempt classification according to their occurrence, although they must be considered in all investigational and special specification work.

MAKE-UP OF CHARGE

The second causative factor is the structural and mechanical make-up of charge. While important, it seems impossible at the present time to attempt more than a qualitative classification. The degree of graphitization of charged materials has some effect on the possibility of degree of solution of graphite during the melting process, hence on the presence of graphitic nuclei in the molten metal. This of course influences the degree of graphitization of the final casting. Thus the amounts of sand-cast pig, chilled pig, steel, scrap, etc., should be noted in all research work. Furthermore the size of the pieces charged, their general grain size, etc., should be indicated. Much careful research is needed to demonstrate to what degree the so-called inherent properties of the raw materials persist unaltered through the melting process and influence the final product.

MELTING PROCESS

The cupola or furnace operation has much to do with the quality of metal melted therein. It is known, for example, that superheating during the melting process at first tends to delay the progress of graphitization on cooling. At still higher temperatures this phenomenon may be altered or changed. Iron badly oxidized in cupola melting does not compare favorably with that obtained from a good hot working cupola with normal blast. Crucible-melted irons generally are not comparable with those obtained from the cupola, a fact that has vitiated the practical significance of the work of many investigators. Therefore in any work on comparison or correlation, the type of furnace and its operating characteristics should be clearly stated.

THERMAL AND MECHANICAL HISTORY

The thermal and mechanical history, from furnace spout to cooled casting is the fourth factor. The degree of graphitization depends largely on cooling velocity from liquidus to a temperature probably somewhat below the eutectoid point. The author regards cooling velocity and composition the most important factors

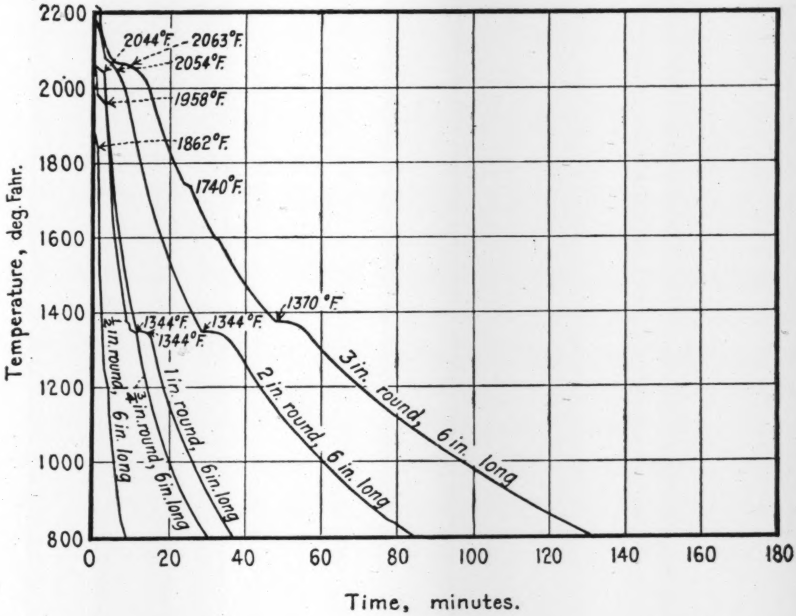

FIG. 1.—Cooling Curves of Cast Iron. Cast in Baked Sand Molds.

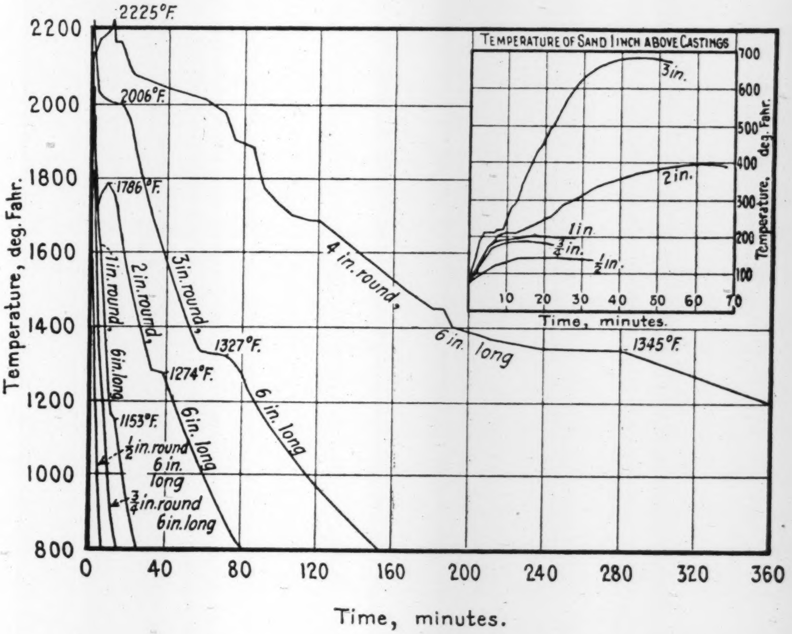

FIG. 2.—Cooling Curves of Cast Iron. Cast in Iron Foundry Green Sand Molds,

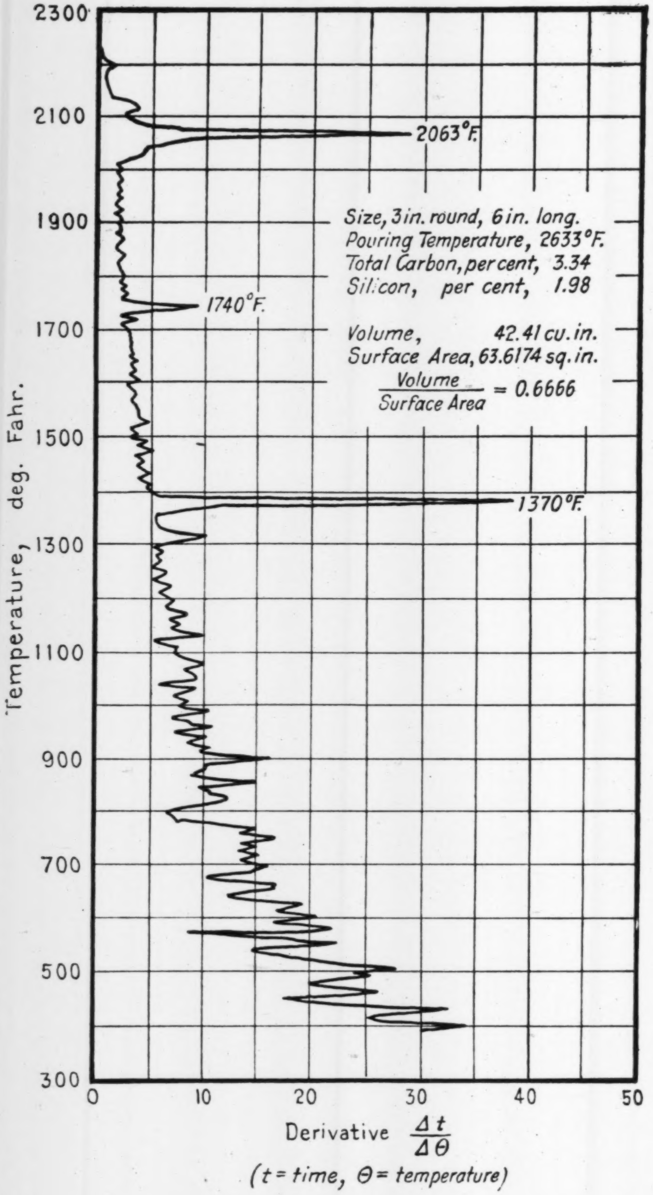

FIG. 3.—First Derivation, $\frac{\Delta t}{\Delta \theta}$ Curve of Cooling Curve of Regular Cast Iron. Cast in Baked Sand Mold.

influencing the final properties of the metal. Cooling velocity is a curvilinear function, not easy to express in a simple manner. (See Figs. 1 and 2.) It depends on many factors. Among these are:

1. Initial temperature (which may or may not coincide with pouring temperature);
2. Final temperature;
3. Specific heat of the alloy, at all points along the curve;
4. Volume of metal to be cooled;

TABLE II.—CAST IRONS ARRANGED IN THE ORDER OF THEIR VOLUME - SURFACE AREA RELATIONSHIP.

Group	Size of Bar	Length Cast, in.	Length Between Supports, in.	Cross-Sectional Area, sq. in.	Surface Area, sq. in.	Volume, cu. in.	Ratio, Volume to Surface Area	Relative Modulus of Rupture	Logarithm of Modulus of Rupture
No. 1	$\frac{1}{4}$ by 12 by 12 in.	..	..	.....	.....	.....	0.1200	....	....
No. 2	0.500 in. diam.	21	18	0.196	33.4	4.12	0.1234	366.640	2.56425
No. 3	0.800 in. diam.	15	12	0.503	38.7	7.55	0.1951	59.672	1.77577
No. 4	0.875 in. diam.	21	18	0.601	58.9	12.6	0.2139	68.414	1.83514
No. 5	$\frac{1}{2}$ by 12 by 12 in.	..	..	.....	....	....	0.2308	......	....
No. 6	1.00 in. diam.	15	12	0.785	48.7	11.8	0.2423	30.552	1.48504
No. 7	1.00 in. diam.	21	18	0.785	67.5	16.5	0.2444	45.831	1.66116
No. 8	1 by 1 in.	27	24	1.00	110.0	27.0	0.2455	36.0	1.55630
No. 9	1.20 in. diam.	21	18	1.131	81.4	23.8	0.2924	26.523	1.42362
No. 10	1.25 in. diam.	15	12	1.227	61.4	18.4	0.2997	15.666	1.19496
No. 11	1.30 in. diam.	21	18	1.327	88.1	27.9	0.3156	20.860	1.31931
No. 12	2 by 1 in.	27	24	2.00	166.0	54.0	0.3253	18.0	1.25527
No. 13	1 by 2 in.	27	24	2.00	166.0	54.0	0.3253	9.0	0.95424
No. 14	1.40 in. diam.	21	18	1.539	95.4	32.3	0.3386	16.702	1.22277
No. 15	1.50 in. diam.	15	12	1.767	74.2	26.5	0.3571	9.052	0.95674
No. 16	1.50 in. diam.	21	18	1.767	102.5	37.1	0.3620	13.579	1.13287
No. 17	1.5 by 1.5 in.	27	24	2.25	166.5	60.75	0.3650	10.67	1.02816
No. 18	1 by 12 by 12 in.	..	..	.....	.....	.....	0.4286	....	....
No. 19	2.00 in. diam.	15	12	3.142	100.5	47.1	0.4686	3.819	0.58195
No. 20	2.00 in. diam.	21	18	3.142	138.2	66.0	0.4776	5.7288	0.75806
No. 21	2 by 2 in.	27	24	4.00	224.0	108.0	0.4820	4.500	0.65321
No. 22	2.20 in. diam.	21	18	3.801	152.7	79.8	0.5226	4.3042	0.63389
No. 23	2.50 in. diam.	21	18	4.909	174.8	103.1	0.5898	2.9332	0.46734
No. 24	2.5 by 2.5 in.	27	24	6.25	282.5	168.75	0.5980	2.303	0.36229
No. 25	3.00 in. diam.	15	12	7.069	155.5	106.0	0.6817	1.1316	0.05369
No. 26	3.00 in. diam.	21	18	7.069	212.1	148.4	0.6996	1.697	0.22968
No. 27	3 by 3 in.	27	24	9.00	342.0	243.0	0.7110	1.333	0.12483
No. 28	2 by 24 by 24 in.	..	..	....	.....	......	0.8571	.....	....
No. 29	4.00 in. diam.	15	12	12.57	213.6	188.6	0.8830	0.477	9.67852-10
No. 30	3 by 24 by 24 in.	..	..	....	.....	.....	1.20	.....	....
No. 31	6.00 in. diam.	15	12	28.27	339.3	424.1	1.250	0.141	9.14922-10
No 32	4 by 24 by 24 in.	..	..	.....	.....	.....	1.440	.....	....
No. 33	8.00 in. diam.	15	12	50.27	477.5	754.1	1.579	0.0596	8.77525-10

5. Metal surface area, for dissipation of heat;
6. Cooling gradients within metal and mold;
7. Conductivity of mold material;
8. Speed of pouring;
9. Energy release apparent in thermal arrests (see Fig. 3); and
10. Mechanical factors.

It is apparent that with carefully regulated foundry practice many of these factors are nearly constant, or at least susceptible to a reasonable degree of control. The most significant factor is the relationship of volume to surface area, expressing the well-known foundry fact that cooling rate is proportional to section size. In Table II is shown a list of various simple shapes arranged in the order of their volume - surface area relationship. In Fig. 4 is presented experimental proof of the validity of this ratio for bars cast under carefully controlled and reproducible conditions—complying with the assumptions dealt with when applying constants indicated in Table II. In this we have used the average rate of cooling from the solidification arrest (about 2050° F.) to the pearlitic inversion (about 1350° F.). That there is

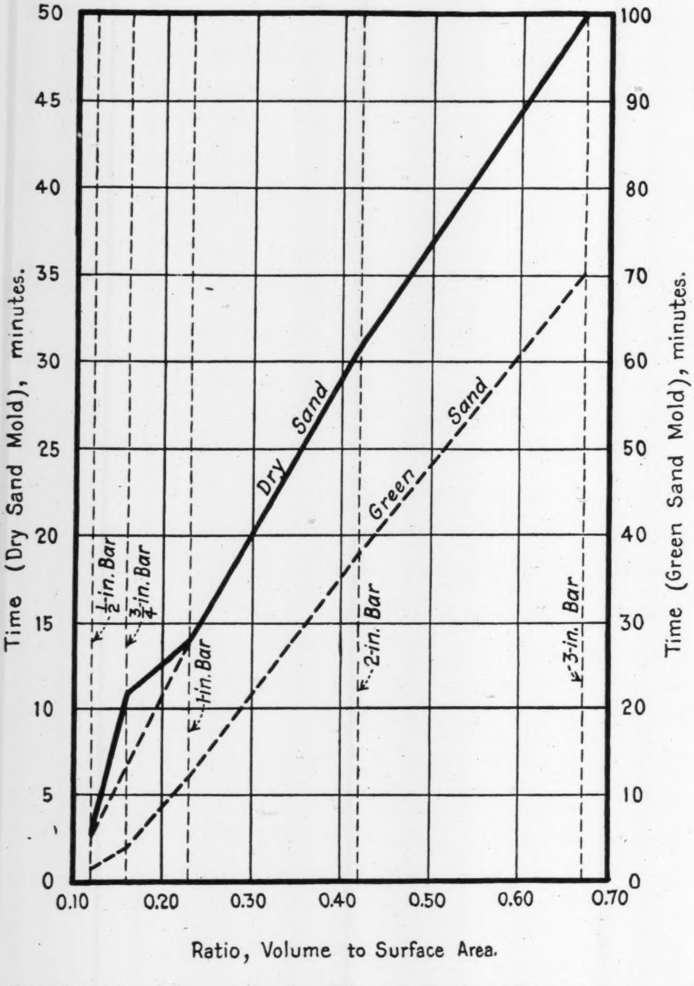

FIG. 4.—Relationship of Actual Cooling Rates to Ratio of Volume to Surface Area. Time in minutes from beginning of solidification to end of entectoid arrest.

some graphitization below 1350° F. is well known. It is interesting to contemplate the net effect of time-temperature or cooling curves *versus* temperature-graphitization rates on the net rate of graphitization under casting conditions. The time-temperature curve is hyperbolic, whereas the temperature-graphitization rate is parabolic.

Initial temperature depends to some degree on the process employed and on the method of operation. Pouring temperature is controllable. Final temperature may be assumed as constant. The specific heats of various types of iron do not vary much.

Magnitude of cooling gradients within the metal itself are as yet little understood. That they are very important is evident from the variations in grain size found on the cross-section of some castings, notably large sections with high-carbon irons.

Conductivity of mold material of course varies somewhat. (See Fig. 4.) This is controllable for research purposes. Speed of pouring depends on gating and workmanship, both relatively controllable, but rarely defined in experimental work.

Mechanical factors often are overlooked. Were the bars sand-cast, were they tumbled, what sort of risers were used, etc.?

In any work of classifying irons according to mechanical test, the method of testing should be clearly and completely defined. In much research work neither analysis nor size of sections is mentioned.

DESIGN AND WORKMANSHIP

The fifth item, design and workmanship, has much to do with the soundness hence service properties of castings. This well-known item is quite thoroughly discussed in many articles on foundry practice, and no further reference will be made here.

[For Discussion on Classification of Cast Iron, see page 142.—ED.]

EFFECT OF SECTION AND VARIOUS COMPOSITIONS ON PHYSICAL PROPERTIES OF CAST IRON

By R. S. MacPherran[1]

Synopsis

This paper describes a study of the effect of section on the tensile strength and hardness of three types of cast iron: a hard low-silicon gray iron, a soft high-silicon gray iron, and a high-test cast iron. Test bars ranging in diameter from 1¼ to 4 in. were cast. Brinell hardness determinations of the entire cross-section of the bars were made, and tensile strength of the material at the center and outer edge of the bars determined.

Whereas the two gray irons were appreciably harder at the sides than at the center, the high-test iron here reported had practically equal Brinell hardness throughout. The tensile strength of this high-test iron decreases as the thickness of bar increases, that is, as the current is retarded, though not to the extent shown by the other two types of iron.

The tests described in this paper were conducted with a three-fold object: first, to determine the effect of increased section on the tensile strength of cast iron; second, to determine the effect on the hardness of the removal of various amounts of stock; and third, to determine if the high-test iron followed the same laws as the weaker grades.

Three types of cast iron were selected for test: a hard low-silicon gray iron, designated as type A; a soft high-silicon gray iron, designated as type C; and a high-test cast iron, designated as type HT. The chemical compositions of these three types as determined by analysis were as follows:

	Total Carbon, per cent	Silicon, per cent	Sulfur, per cent	Phosphorus, per cent	Manganese, per cent	Nickel, per cent
Type A........	3.60	1.29	0.121	0.21	0.78	....
Type C........	3.42	2.23	0.084	0.54	0.70	....
Type HT.......	2.55	2.23	0.096	0.08	0.86	1.43

All bars of any one grade were cast from one ladle of iron. All specimens were molded as required for the arbitration test bar (round, in cores and cast vertically, gated on top, 15 in. long). The specimens, as cast, were sawed in two in the middle of the bar. The cross-sections

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[1] Chief Chemist, Allis-Chalmers Manufacturing Co., West Allis, Wis.

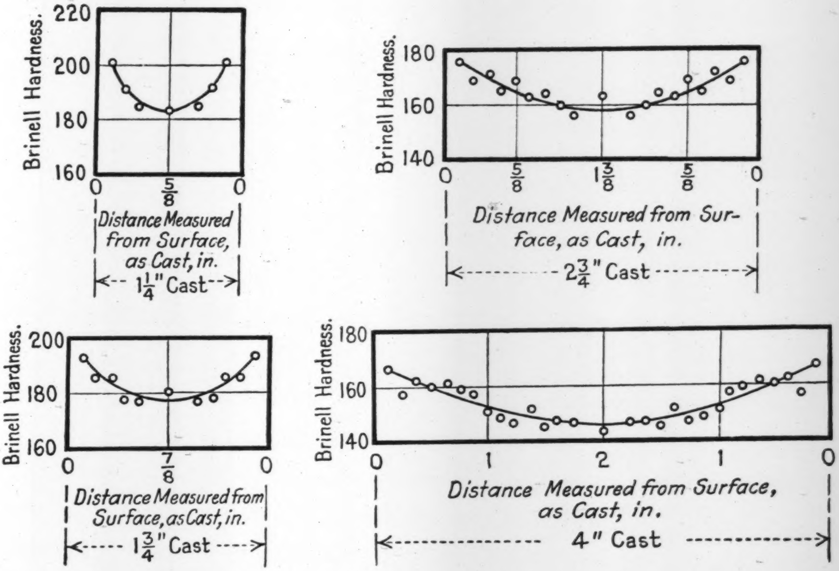

Fig. 1.—Measurement of Brinell Hardness for Different Thicknesses of Hard Gray Iron (Type A).

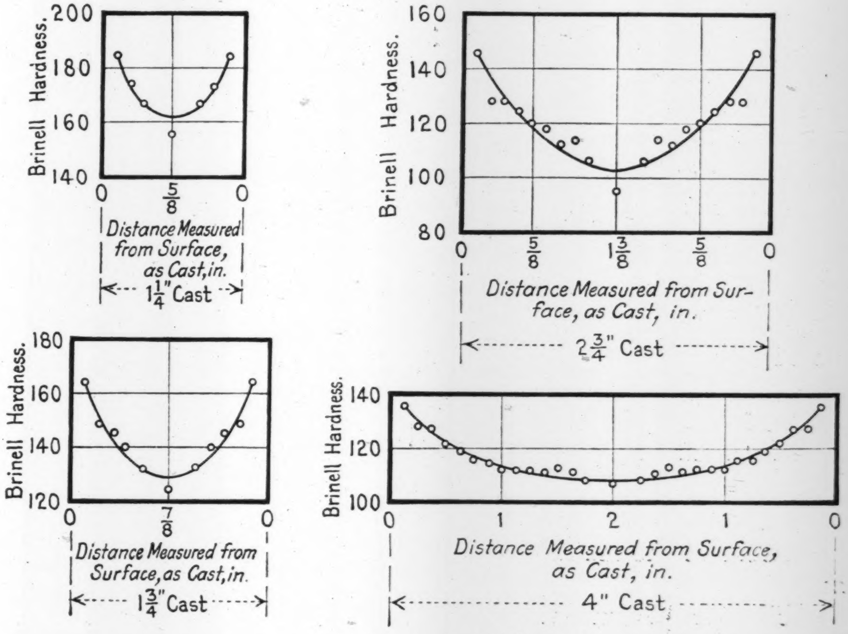

Fig. 2.—Measurement of Brinell Hardness for Different Thicknesses of Soft Gray Iron (Type C).

were polished and explored for Brinell hardness. This was done by setting one end of a pair of dividers in the center of the section and drawing concentric circles $\frac{1}{8}$ in. apart out to the outer edge of the specimen. The hardness was then determined 90 deg. apart on these circles. All reported tests for hardness are, therefore, averages of at least four determinations.

Tension test specimens were machined from the lower half of the test bars to 0.505 in. in diameter in the center with $\frac{3}{4}$-in. diameter threaded ends. To determine the differences in strength, if any, between metal at the center and near the circumference of the bars, one set of tension specimens was machined from the center of the original bar and one set from the outer 1 in. of the original bar.

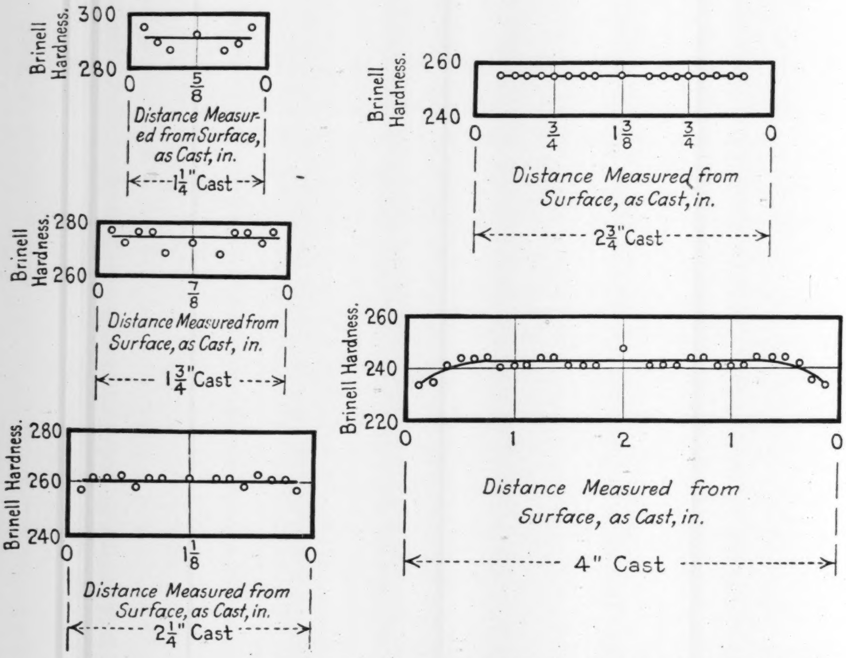

FIG. 3.—Measurement of Brinell Hardness for Different Thicknesses of High-Test Cast Iron (Type HT).

HARDNESS TESTS

Results of the hardness determinations are plotted in Figs. 1, 2 and 3.

The effect of removal of stock is shown in the falling off in the hardness of types A and C as the surface is removed. In type C, for

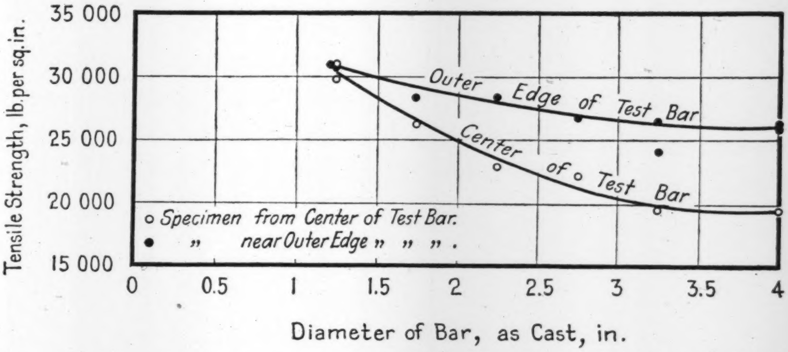

FIG. 4.—Relation Between Tensile Strength and Thickness of Section as Cast, for Hard Gray Iron (Type A).

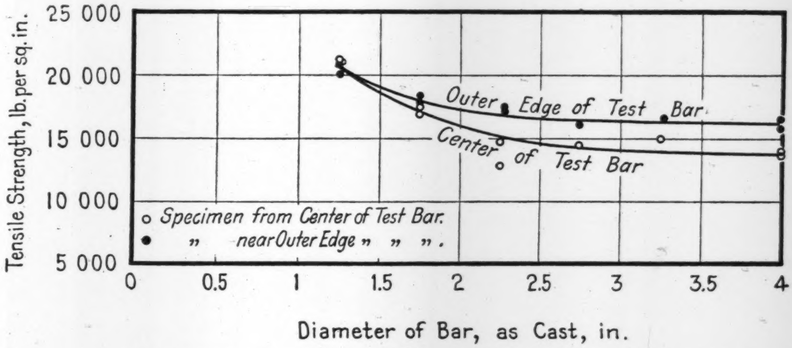

FIG. 5.—Relation Between Tensile Strength and Thickness of Section as Cast, for Soft Gray Iron (Type C).

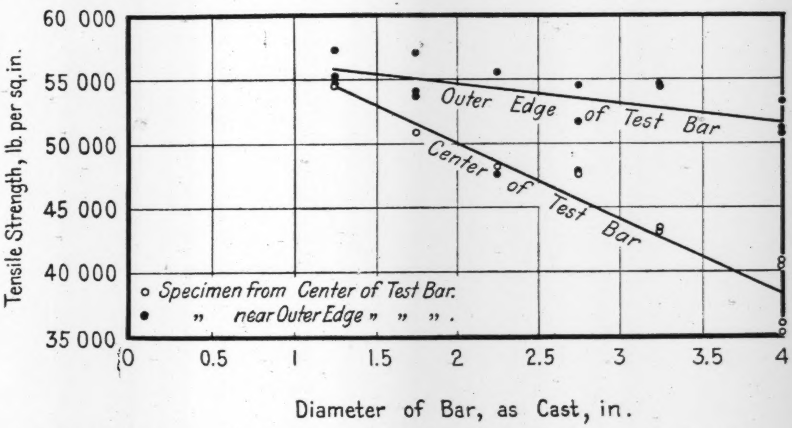

FIG. 6.—Relation Between Tensile Strength and Thickness of Section as Cast, for High-Test Cast Iron (Type HT).

example, it will be noted (Fig. 2) that, assuming $\frac{1}{8}$ in. stock removal as normal, the removal of the next $\frac{1}{8}$ in. results in a drop of from 10 to 15 points in Brinell hardness. Another $\frac{1}{8}$ in. would mean an additional loss. In type A, the drop from the first $\frac{1}{8}$ in. to the second $\frac{1}{8}$ in. is about 10 points in Brinell hardness.

On the high-test iron, type HT (Fig. 3), an interesting development was that the outer $\frac{1}{8}$ in. of the larger size bars is softer than the iron farther from the surface. When once the outer layer is removed, however, the remainder of the section seems to be of uniform hardness. This effect has been confirmed by a number of coupon bars cast on large castings which, on removal, show a softer outer $\frac{1}{8}$ in. than is shown by the body of the castings.

TENSILE STRENGTH

The values of tensile strength for the three types are plotted in Figs. 4, 5 and 6, showing the relation between diameter and test bar as cast and the tensile strength of the iron at the center of the bar and near the outer edge of the bar. Data were reported to the Society a year ago[1] on irons very similar in composition to those reported upon here. The present investigation includes bars up to 4 in. in diameter and the complete data are reported in this paper.

The high-silicon iron, type C, shows a greater proportionate loss in strength in increasing from the smaller sizes to the medium sizes, but shows only a slight loss in going from medium to the larger sizes. This may be due to the fact that in the medium sizes the structure becomes practically ferrite, which would not be materially weakened by additional slower cooling. Types A and HT, however, would still have considerable pearlite at the medium sizes and consequently would show a further weakening as the time of cooling is increased in going to the larger sections.

The high-test iron shows less falling off in strength as compared to the other grades. This is particularly true of the bars taken from the outer 1 in. of casting (see Fig. 6). The tensile strength of iron near the outside of the 4-in. bar of high-test cast iron is only 7.2 per cent less than the tensile strength of the iron from the outer part of the $1\frac{1}{4}$-in. bar, as compared with a corresponding loss of 16 per cent in type A iron and 21 per cent in type C iron. It need hardly be pointed out that the outer 1 in. in a 4-in. round bar represents three-fourths of the entire area of the bar. This feature, therefore, is decidedly in favor of the high-test iron.

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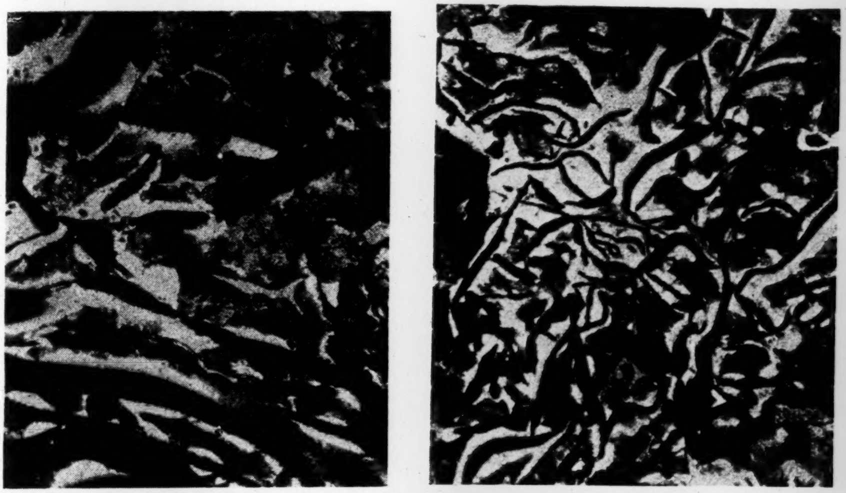
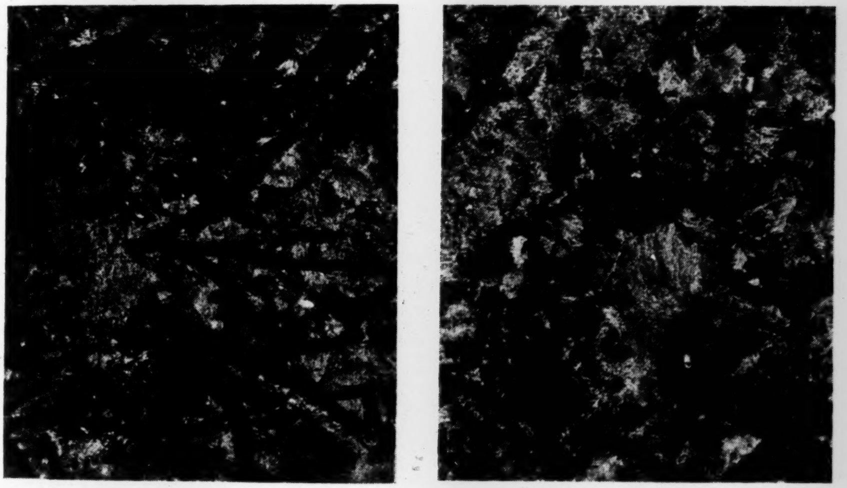
[1] See Discussion of Report of Committee A-3 on Cast Iron, *Proceedings*, Am. Soc. Testing Mats., Voi. 28, Part I, p. 144 (1928).

In Fig. 7 are shown photomicrographs of the structure of soft gray iron, type C, and high-test cast iron, type HT, at the center and outer edge of the 4-in. bars. It will be noted that the structure of the high-test iron becomes open as the center of the casting is approached.



Center Outer Edge

Soft Gray Iron (Type C).



Center Outer Edge

High-Test Cast Iron (Type HT).

Fig. 7.—Photomicrographs of Structure of Two Cast Irons from Center and Outer Edge of 4-in. Rounds. Etched in Picric Acid (× 100).

CONCLUSIONS

The following conclusions are justified:

The effect of increasing the section on the types A and C iron is to materially and progressively lower the tensile strength.

Removal of extra stock in these types is progressively accompanied by loss of hardness.

Under cooling conditions similar to those described, the high-test iron here reported upon has practically equal Brinell hardness throughout the section.

The grain of the high-test iron is somewhat more open as the center of the casting is approached. (See micrograph.)

That the tensile strength of the high-test iron tends to fall off as the section is increased or as the cooling is retarded, though not to the extent shown by the other two types.

[For Discussion on Properties of Cast Iron, see page 142.—ED.]

CORRELATION OF CAST-IRON TEST BARS AND CASTINGS

By W. H. Rother[1] and V. M. Mazurie[2]

Synopsis

This paper reports briefly the work undertaken by various investigators in attempting to correlate the properties of cast-iron test bars with the properties of the casting. Few really tangible facts have been established.

The discussion calls attention to the following features:

The investigstors agree that more than one size of test bar is necessary where any account is to be taken of the variation of strength with thickness.

Cast iron as produced in the modern foundry is really a series of alloys. Any attempt at correlation of test bar with casting is futile unless this fact is recognized.

It is believed that much can be accomplished by casting test bars having the same relative cooling rate as the casting. Work along this line will be undertaken in the near future.

A study of the variation in strength of cast iron with section should be of great assistance to designers of cast-iron structures. At the present time, we determine the "quality of the iron in the ladle" from the standard arbitration bar. The consumer is interested only in his casting and is little impressed by the statement of such a vague property. It is of paramount importance to the foundryman that some test procedure be developed wherein the properties of the test bar can be correlated with the properties of the casting. This is especially so for castings required to withstand high water and steam pressures and those working under severe strains and shocks.

In steel and non-ferrous foundries, the tension test is generally accepted as being representative of the casting. The properties determined by this test can be compared, fairly accurately, with the properties of the casting. Due in part to the complex nature of cast iron, the foundryman favors the transverse test. In this, however, he has a measuring rod which measures in units difficult to tie up with the properties of a casting. Until he is able to correlate the casting with the test, he is working at a disadvantage. In this paper a brief résumé of work done on this subject during the past few years is given.

Studies of Correlation of Test Bar and Castings

One of the authors[3] pointed out the variation in tensile strength found with different size sections and suggested the use of more than one size of transverse test bar in order that the structure found in castings of varying thickness might be truly represented. An intelligent choice of the proper size of test bar is necessary, as well as a true appreciation of the variations caused by different foundry practice. British foundrymen have for some years advocated three standard test bars of small, medium and large diameters to represent castings of the corresponding three classes.[1]

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[1] Metallurgist, Buffalo Foundry and Machine Co., Buffalo, N. Y.

[2] Assistant Metallurgist, Buffalo Foundry and Machine Co., Buffalo, N. Y.

[3] W. H. Rother, "Strength of Cast Iron and Thickness," *The Iron Age*, August 7, 1924.

In 1926 the authors[2] attempted to correlate the transverse test with the casting by testing to failure round bars, varying in diameter from 1 to 3 in., in increments of ½ in. Using the strength of a 1-in. bar as a basis of reference, theoretical strengths were computed for bars larger in size. By comparing the theoretical strength with the actual strength, a relation was obtained between castings of large and small sections. When tested transversely to failure, three iron mixtures showed the following relations:

A common iron (2.15 per cent silicon) showed, for each increase of ½ in. in diameter, an average decrease in strength of approximately 5.80 per cent.

An iron containing an addition of 15 per cent of steel (1.70 per cent silicon) showed, for each decrease of ½ in. in diameter, an average decrease in strength of approximately 4.20 per cent.

An iron containing an addition of 25 per cent of steel (1.15 per cent silicon) showed, for each increase of ½ in. in diameter, an average decrease in strength of approximately 4.35 per cent.

With increases up to about 2 in., the strength changes rapidly. Above 2 in. the variations in average structure are not so great, and the decrease of strength with increase in dimension, is not so pronounced.

In sets of square bars cast with the corresponding round bars, it was found that bars cast and tested square gave lower specific transverse strengths than round bars.

In this series of tests it was hoped that some simple relationship might be evolved that would enable one to compute the strength of any size section from the strength of a small size test bar. While a rough comparison may be drawn in this manner, it is not practicable to include all iron under one group.

A paper published in 1926 by Talbot and Richart[3] gives interesting data on cast-iron pipe. In these tests sample test specimens were cut from pipe sections and their properties compared with the properties of test bars cast at the same time as the pipe. Bursting, flexure and impact tests of the pipe itself made possible quite valuable comparisons of the properties of test specimen, test bar and casting.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part I, p. 215 (1924).

[2] W. H. Rother and V. M. Mazurie, "Strength of Cast Iron in Relation to Its Thickness," *Transactions*, Am. Foundrymen's Assn., Vol. 34, p. 746 (1926).

[3] A. N. Talbot and F. E. Richart, "A Study of the Relation Between Properties of Cast-Iron Pipe Tested Under Impact, Internal Pressure, and Flexure, and the Corresponding Properties Found in Several Kinds of Test Specimens Taken Therefrom," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 185 (1926).

In the Appendix to the report of Committee A-3 on Cast Iron for 1928[1] the findings of Sub-Committee XIV on Correlation of Test Bar and Castings are given. The work of this sub-committee was carried on in practically the same manner as that done by the authors in 1926 and the values obtained agree closely with those found by the authors.

In a paper published in 1928, J. G. Pearse[2] recommends consideration of the modulus of rupture as a criterion of transverse strength. In this connection it was shown that a regular relationship exists between the modulus of rupture and the diameter for bars of the same composition and method of casting. As the diameter of section increases, the modulus of rupture decreases. For comparing sections of the same size and shape, the modulus of rupture is a good criterion. For comparing sections of different sizes, it is not particularly applicable, unless some account is taken of the differences in size.

In a paper published in 1928 by Bolton[3] a method of correlating the test bar with the casting is proposed, which is based on the relative cooling rate. "The relative cooling rates are roughly proportional to the ratio of volume to surface area, or, in other words, proportional to the section size." The proposed method of correlation can best be illustrated by citing an example.

In studying a casting one would first calculate the ratio of volume to surface area for the casting and then select a test bar having the same ratio of volume to surface area. By casting the test bar from the same metal as the casting, a close approximation of the physical properties of the casting would be obtained, as both casting and test bar would have the same relative cooling rate.

This method, while involving considerable calculation and a fine perception of the variables of foundry practice, approaches the problem in a very logical manner. This predicts for future work a variety of sizes in test bars, a round bar being preferred.

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[1] Report on Correlation Tension and Transverse Tests of Cast Iron, *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 142 (1928).

[2] J. G. Pearse, "The Use and Interpretation of the Transverse Test for Cast Iron," *Transactions*, Iron and Steel Inst., British (1928).

[3] J. W. Bolton, "On Research Problems of the Gray Iron Foundry," *Transactions*, Am. Foundrymen's Assn., Vol. 36, p. 469, (1928).

DISCUSSION

This, in a brief and sketchy form, gives an idea of the importance of the problem and the methods of approach attempted. While everyone seems to appreciate the desirability of correlating casting with test, there appears to be doubt as to just how this may be accomplished. Objections have been made to both the tension and transverse test as conducted at the present time.

While not used to a great extent, the tension test offers possibilities which should not be overlooked. This test measures properties which may be applied readily in interpreting the value of a casting. By making due allowance for the various factors affecting the test, valuable data may be determined.

The transverse test is looked upon with favor by the foundrymen, but this too has features which are objectionable. However, if the problem is to be solved in a simple manner, this appears to be the test to be correlated with the casting.

CONCLUSIONS

In order to arrive at any logical conclusions, it must be understood that gray iron is not an alloy, but a series of alloys. In the modern foundry it is possible to make iron of any strength between 20,000 and 55,000 lb. per sq. in. So, too, the transverse strength of bars broken on a 12-in. span may be varied between 2500 and 5500 lb. Shearing strength, Brinell hardness, and compressive strength may be varied between wide ranges. Naturally for different strengths a different mixture is necessary. It is apparent then that gray iron cannot be classified as a single metal but as a series or group of metals. After systematically classifying the different grades of iron, the problem then is to find out what these different grades will do in the casting.[1]

Through a proper interpretation of the cooling rate of iron it is felt that the physical properties of a test bar can be correlated with the properties of the casting. This does not imply that a single size bar will be applicable to every case. It is believed that specifications in the future will cover test bars of several sizes, and that with an intelligent choice of test bar the properties of a casting will be predicted accurately from the properties of the test bar.

[For Discussion on Properties of Cast Iron, see page 142.—ED.]

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[1] For a discussion of this subject see also paper by J. W. Bolton on "Classification of Gray-Iron Alloys," p. 67.

STATIC STRENGTH OF PLAIN AND ALLOY CAST IRON

By F. B. Coyle[1]

Synopsis

The author states his opinion that the only tests for strength of cast iron which give absolute results are those for tension and compression.

The Maurer diagram showing relation between structure and composition of cast iron is given. An extension of this diagram to show the relation of composition and structure to tensile strength is presented, followed by a series of diagrams showing the extension of the range of composition which will produce a pearlitic structure and maximum strength.

Static strength in relation to the testing of materials refers to the strength when determined upon testing machines when the movements of the elements of the machine are at relatively slow speeds. It must be borne in mind that the term strength, when considered by designers, is used in the absolute sense. The only tests for strength of cast iron which are absolute are those for tension and compression. For this reason the author is presenting herein some data regarding the tensile strength only of cast iron.

In 1927 the author published some data relative to the strength and properties of cast iron for Diesel engines.[2] In that paper was reproduced a diagram showing the relation between composition and structure of cast iron when cast under the condition designated by the American Society for Testing Materials for the standard arbitration test bar.[3] The author has since carried the work a step farther and demonstrated the relation between composition, structure and strength of cast iron. It is the aim in the present paper to carry this work still farther by not only showing the further modification of Maurer's original diagram for plain iron but the modification of the strength diagram as well by the addition of alloying elements. Maurer's diagram and the author's diagram have already been presented before this Society[4] and for convenient reference are reproduced here. Figure 1 is a reproduction of Maurer's original diagram.[5]

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[1] Research Metallurgist, International Nickel Co., Bayonne, N. J.

[2] F. B. Coyle, "Properties and Heat Treatment of Cast Iron for Diesel Engines," *Transactions*, Am. Soc. Steel Treating, September, 1927.

[3] See Standard Specifications for Gray-Iron Castings (A 48 – 18), 1927 Book of A.S.T.M. Standards, Part I, p. 441.

[4] Discussion by F. B. Coyle of paper by M. E. Greenhow on "High-Test Alloy Cast Iron," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 84 (1927).

[5] Maurer, *Kruppsche Monatschefte*, July, 1924.

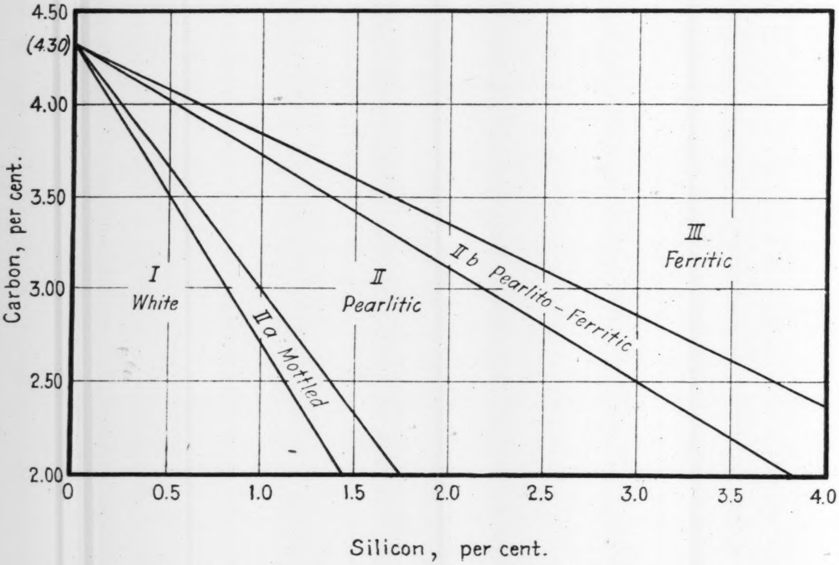

FIG. 1.—Constitutional Diagram of Cast Iron (Maurer).

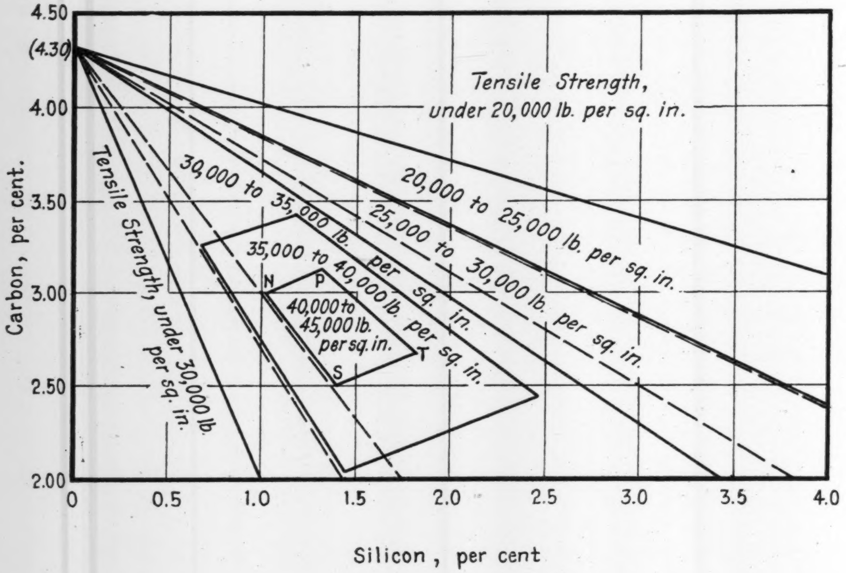

FIG. 2.—Constitutional and Strength Diagram of Cast Iron.

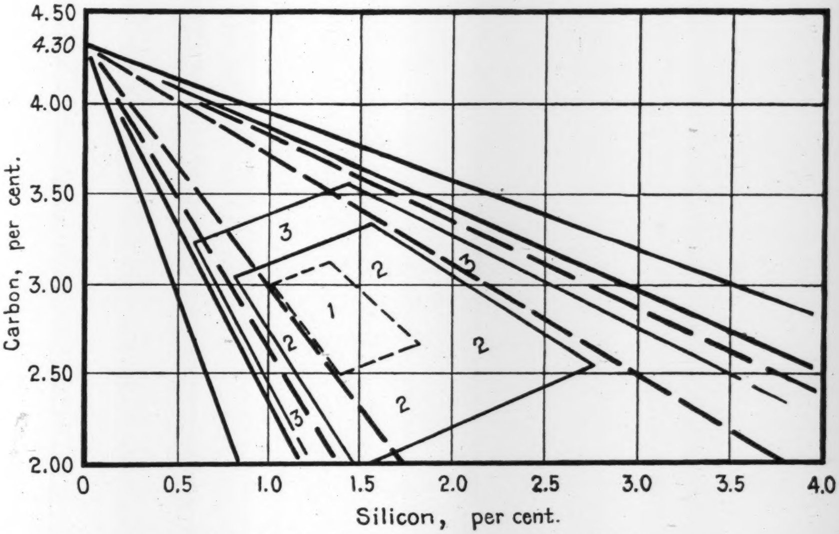

FIG. 3.—Constitutional and Strength Diagram of Cast Iron Containing 2.0 per cent of Nickel.

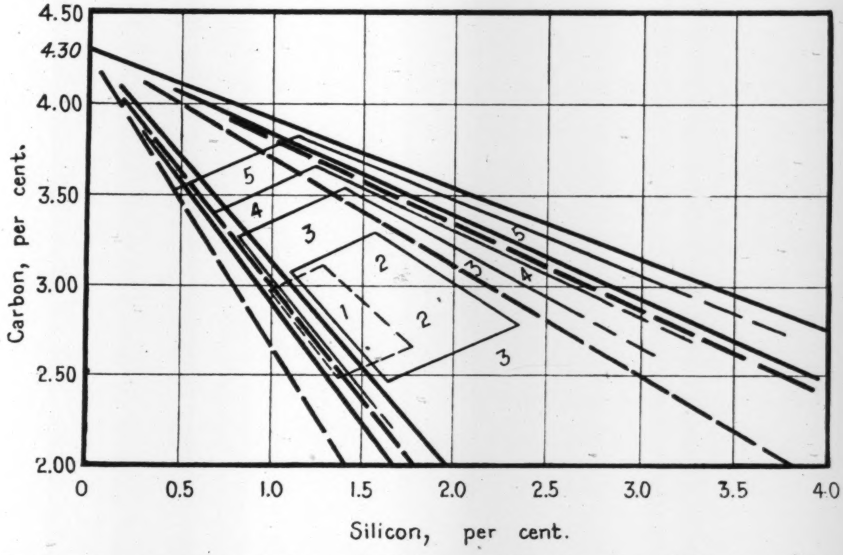

FIG. 4.—Constitutional and Strength Diagram of Cast Iron Containing 0.50 to 0.75 per cent of Chromium.

The correctness of this diagram below 4.0 per cent of carbon has not been established by the author only but also by several other investigators. The region of this diagram above 4.0 per cent of carbon has been debated, but since the region below 4.0 per cent carbon is all that is of industrial value this uncertainty can be safely disregarded. There are five zones marked off which show the relation of composition to the structure of arbitration test bars; these can be summarized as follows:

Zone	Structure
I	White
II*a*	Mottled
II	Pearlitic
II*b*	Pearlito-ferritic (pearlite predominant over ferrite)
III	Ferritic (ferrite predominant over pearlite)

In Fig. 2 is reproduced the author's strength diagram of plain cast iron above referred to, showing the relation between composition, structure and strength. This diagram was constructed from the results of over 3000 tension tests conducted over a period of ten years. In 1927 a similar diagram was published by a German investigator[1] which showed quite different results. However, this diagram was constructed from the results of only ten tests, which seem too meager to justify drawing broad conclusions. The correctness of Fig. 2 has been substantiated since its publication by at least six other individuals. This work has been carried farther by studying the effect of the alloying elements nickel and chromium upon these two diagrams. The results are presented in Figs. 3, 4, 5 and 6. Beside his own data the author has included data published by T. H. Wickenden and J. S. Vanick, E. Piwowarsky, and R. Moldenke. In the construction of Figs. 3, 5 and 6 the results of about 500 tests were used; in that of Fig. 4 about 150 tests.

Figure 3 shows the modifications produced in Figs. 1 and 2 by the addition of 2.0 per cent of nickel. The heavy broken lines indicate Maurer's original diagram; the heavy solid lines indicate the modification of that diagram. The zone of white iron is reduced one third due to the graphitizing action of nickel. On the other hand the combined carbon in pearlite is stabilized as shown by the increased zone of pearlitic compositions. The "highest strength" area, *NPST*, of Fig. 2 is superimposed on Fig. 3, as shown by the light broken lines bounding Area 1. Area 2 of Fig. 3 shows how the corresponding area of maximum strength (40,000 lb. per sq. in. and over) is extended over

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[1] P. Kleiber, "Über den Kruppschen Sternguss," *Kruppsche Monatshefte*, June–July, 1927, p. 109.

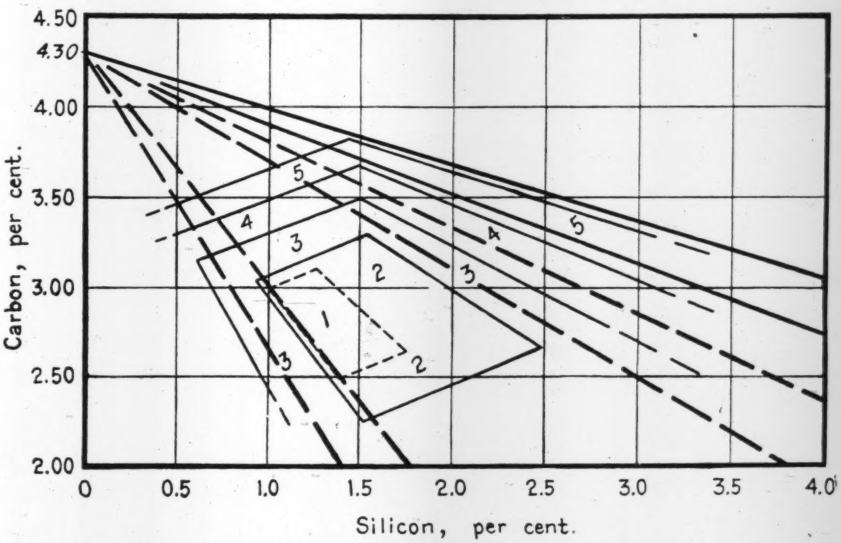

FIG. 5.—Constitutional and Strength Diagram of Cast Iron Containing 0.25 to 0.50 per cent of Nickel and 0.25 to 0.50 per cent of Chromium.

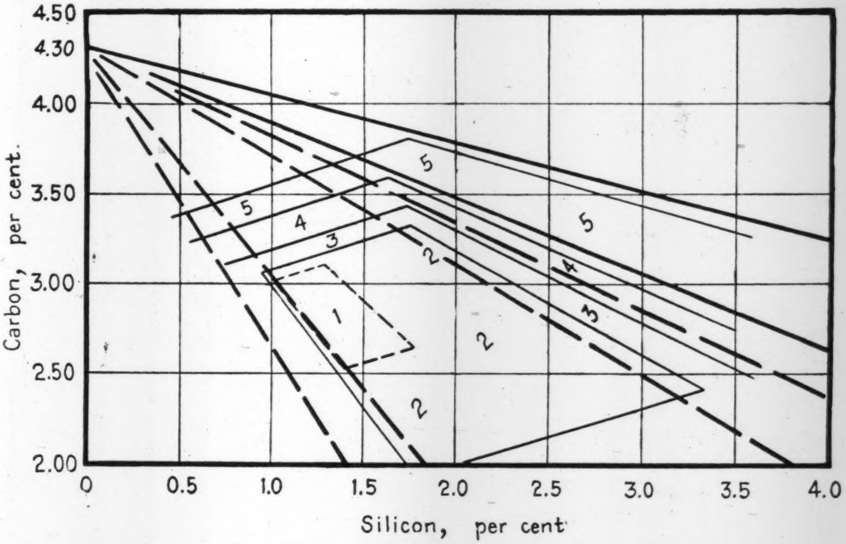

FIG. 6.—Constitutional and Strength Diagram of Cast Iron Containing 0.50 to 1.25 per cent of Nickel and 0.25 to 0.75 per cent of Chromium.

a much wider range of compositions by the addition of 2.0 per cent of nickel. Area 3 indicates, so far as the data available permitted, the range of composition showing tensile strength between 35,000 and 40,000 lb. per sq. in.

In each of the three succeeding Figs. 4, 5 and 6, Area 1 shows the position of the high strength area, *NPST*, of Fig. 2. Area 2 shows the corresponding range of composition for maximum strength for the respective alloy additions; and Areas 3, 4 and 5 in each figure, so far as could be determined by the data available, show range of composition for the following tensile strength ranges:

AREA	RANGE OF TENSILE STRENGTH
3	35 000 to 40 000 lb. per sq. in.
4	30 000 to 35 000 " " " "
5	25 000 to 30 000 " " " "

Figure 4 shows the modifications produced in Figs. 1 and 2 by the addition of 0.50 to 0.75 per cent of chromium. As before, the heavy broken lines indicate Maurer's original diagram and the heavy solid lines the modifications produced by chromium. This element, through its carbide-forming tendencies, has increased the range of compositions which produce white iron. The zone of pearlitic iron is also increased. Area 2 shows that the range of composition producing maximum strength (40,000 lb. per sq. in. and over) is slightly increased; and while eliminated in the lower range of silicon, it is extended appreciably into the range of higher silicon.

Figure 5 shows the modifications produced in Figs. 1 and 2 by the addition of 0.25 to 0.50 per cent of nickel and 0.25 to 0.50 per cent of chromium. The original Maurer diagram is indicated by heavy broken lines. The zone of white iron is unaltered, but the zone of pearlitic iron is extended considerably. The area of maximum strength (Area 2) is extended appreciably into the ranges of higher silicon and lower carbon.

Figure 6 shows the modifications produced in Figs. 1 and 2 by the addition of 0.25 to 0.75 per cent of chromium and 0.50 to 1.25 per cent of nickel. The proportion of nickel to chromium varied from 1¼ of nickel to 1 of chromium to 2 of nickel to 1 of chromium. The alterations of the Maurer diagram are the same as shown in Fig. 5. The area of maximum strength (Area 2) is extended slightly into the range of higher carbon and considerably into the ranges of higher silicon and lower carbon.

CONCLUSION

There has been presented a series of diagrams showing the effect of composition, particularly the elements nickel and chromium, on the tensile strength and structure of cast iron. However, these diagrams should not be considered a sole criterion on the production of castings Other practical modifying characteristics, such as hardness and tendency to chill, should be considered as well as strength in the production of suitable castings.

[For Discussion on Properties of Cast Iron, see page 142.—ED.]

ELASTIC PROPERTIES OF CAST IRON

By J. T. MacKenzie[1]

Synopsis

Elasticity is that property of a body which enables it to return to its former shape after undergoing strain. Cast iron is characterized by a stress-strain relationship represented by a curved plot from origin to breaking point. Formulas for the modulus of elasticity and for the deflection or elongation of cast iron are given.

Cast iron shows pronounced plastic deformation on the first few applications of load. On subsequent applications of the same stress it appears to be fully elastic, although higher stress will cause additional flow, which will disappear after a few loadings at this stress.

The results of some of the author's work are shown presented in the form of graphs. The curves show striking correspondence between amount of "set" (plastic deformation) and amount of graphite. This suggests the probability that a large part of the set is due to some crushing of graphite flakes or to squeezing of iron grains into the flakes or the voids surrounding them. Also, it is shown that the "ultimate" modulus of elasticity is an index of the resistance to plastic deformation.

Elasticity is that property of a body which enables it to return to its former shape after undergoing strain. The modulus of elasticity, in tension or compression, is the quotient of stress divided by linear strain; in bending, it is a function of the load divided by the deflection. The modulus of rigidity is the stress-strain ratio in torsion or shear, which in turn is a function of the modulus of elasticity and of Poisson's ratio.

Cast iron is characterized by a stress-strain relationship represented by a curved plot from origin to breaking point. This plot may show one, two, or even three decided inflections, but never a sharp break. The modulus of elasticity is therefore not a constant, but an exponential function of the load, the exponent being greater than unity. Bach[2] developed the equation:

$$E = C\,\frac{P^m}{e} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

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[1] Chief Chemist, American Cast Iron Pipe Co., Birmingham, Ala.

[2] Bach-Baumann, "Elasticity and Strength."

in which

E = modulus of elasticity,

P = stress,

m = a number greater than unity,

e = elongation, compression, or deflection, and

C = a constant of the condition of loading and of the nature of the iron.

TABLE I.—TRANSVERSE MODULUS OF ELASTICITY FOR VARIOUS SECTIONS OF CAST IRON ON THREE DIFFERENT SPANS, SHOWING DECREASE OF MODULUS WITH DECREASING SPAN.

(Calculated from Adamson and Bell, *Carnegie Scholarship Memoirs*, 1927.)

		Modulus of Elasticity, lb. per sq. in.		
Section	Size. in.	36-in. Span	18-in. Span	12-in. Span
Rectangle......	2 by 1	9 500 000	8 400 000	5 800 000
Square.........	1 by 1	11 500 000	8 900 000	6 400 000
Round.........	0.9	13 900 000	11 200 000	7 400 000
Round.........	1.2	14 400 000	8 900 000	5 800 000
Round.........	2.2	9 600 000	6 800 000	3 900 000

TABLE II.—ANALYSIS OF CAST IRONS TESTED BY AUTHOR.

Iron	Total Carbon, per cent	Graphitic Carbon, per cent	Combined Carbon, per cent	Silicon, per cent	Sulfur, per cent	Manganese, per cent	Phosphorus, per cent
No. 797...........	4.24	3.36	0.88	1.90	0.08	0.42	0.23
No. 829...........	3.39	2.76	0.63	1.67	0.08	0.43	0.69
No. 845...........	2.74	2.01	0.73	2.69	0.10	0.80	0.28
No. 881...........	2.27	1.53	0.74	2.46	0.19	0.71	0.20
No. 840...........	2.72	0.38	2.34	1.07	0.10	0.54	0.14

Sugimura[1] proposed a hyperbolic formula for elongation or deflection:

$$e = \frac{P}{a - bP} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which e and P are as above, and a and b are constants of the particular material and of the conditions of loading. This value for e when substituted in the conventional formulas for E gives remarkably accurate results.

Care must be used in applying modulus calculations to consider the effect of span. The figures in Table I calculated from the paper of Adamson and Bell[2] show conclusively the decrease of modulus with decrease in span. As all of the different bars were poured from the same ladle, the results seem to be beyond question.

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[1] I. Sugimura, "The Elasticity of Cast Iron at Ordinary and Elevated Temperatures," Society of Mechanical Engineers of Japan, August, 1926.

[2] *Carnegie Scholarship Memoirs*, 1927.

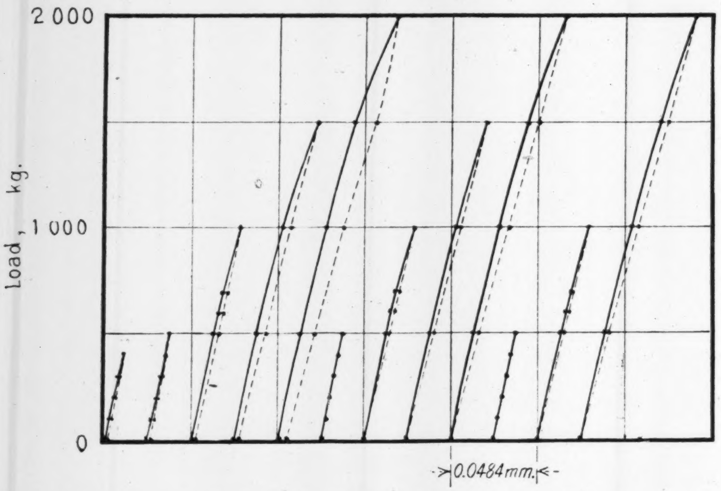

FIG. 1.—Curves Showing Results of Repeated Loading on Specimen in Tension.
Gage length, 7.6 cm. Diameter, 1.904 cm.

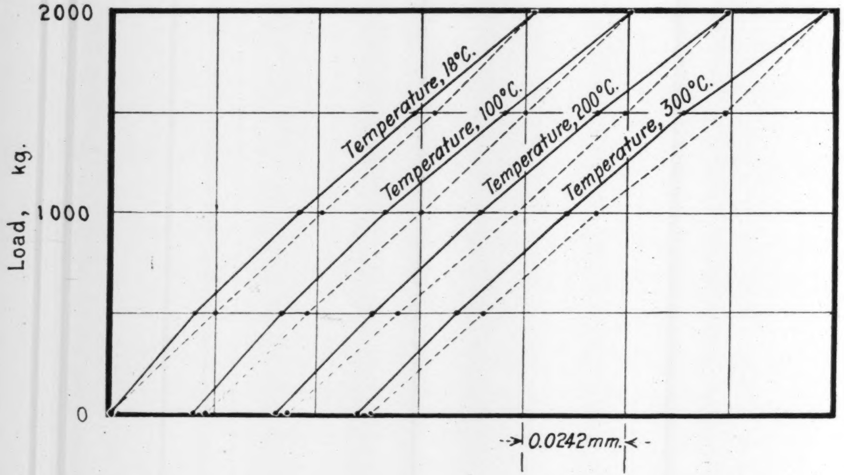

FIG. 2.—Curves Showing Results of Tests on Specimen Used in Fig. 1 at Ordinary and at Elevated Temperatures.
Gage length, 12.7 cm.

Cast iron shows on the first few applications of load pronounced plastic deformation. This does not seem to increase on later applications, and the cast iron appears to be fully elastic on subsequent applications of the same stress. The application of any higher stress will cause an additional flow, which will likewise disappear after a few loadings at the higher stress. If the temperature be raised, there will be further flow which will disappear on further loadings up to approximately three-fourths of the strength and some 400° C. Beyond this temperature the cast iron investigated by Sugimura—machinery cast iron, analysis not given—seems to show plastic flow. Figure 1 shows a series of loading on one bar at ordinary temperature with extensometer readings for loading and unloading as shown. The same bar was then loaded to 2000 kg. at 100, 200 and 300° C., which loadings are shown in Fig. 2, thus proving that with increasing temperature the "plastic" flow again sets in. The scale for elongation is increased to show the flow in greater detail.

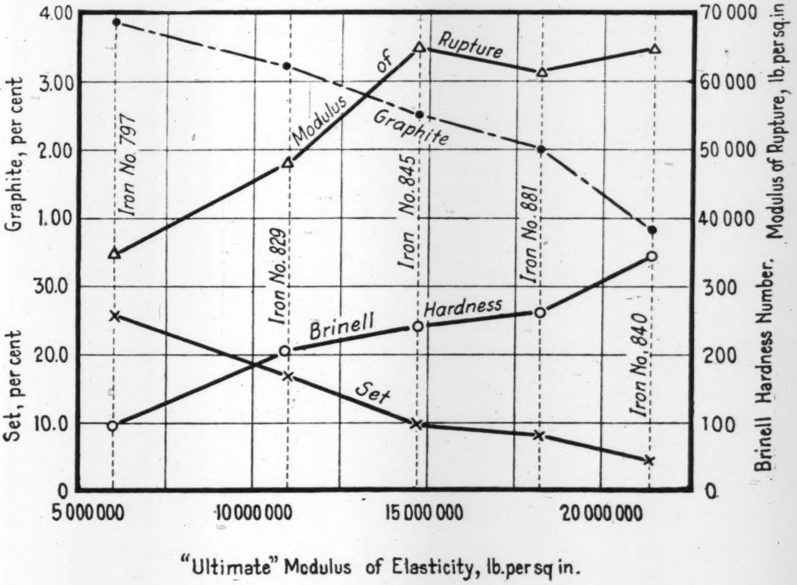

Fig. 3.—Values for Plastic Deformation (Set) in Six Repeated Loadings to Three-Fourths Ultimate Strength Plotted Against Ultimate Modulus of Elasticity, with Trend of Strength, Hardness and Graphite Indicated.

Since neither Bach nor Sugimura give the analysis of the metal used, the author prepared five cast irons differing widely in character, the chemical compositions being recorded in Table II. Sets of bars of each iron were subjected to the usual transverse breaking test and to repeated bending. Three bars were tested transversely to rupture in the usual manner; one was loaded with increasing loads—750, 1000, 1250 lb., etc.—and a complete load-deflection curve obtained for each loading from zero; and one was loaded transversely six times to approximately three-fourths of its ultimate breaking load and the "set" determined. Brinell hardness was also determined. The value of modulus of rupture, hardness, plastic deformation (set) after the sixth loading, and percentage of graphite are plotted in Fig. 3 against the "ultimate" modulus of elasticity, calculated by the conventional beam formula using load and deflection at rupture.

The striking similarity in the curves for "set" and for graphite suggests the probability that a large part of the set is due to some crushing of graphite flakes or to squeezing of iron grains into the flakes or the voids surrounding them. The figure also shows that the "ultimate" modulus of elasticity is an excellent index of the resistance to plastic deformation.

Previous investigators found that three or four loadings were sufficient to give a constant value for deformation under subsequent loadings; Bach, however, reports no more than twenty repetitions of load and Sugimura shows but three, although he mentions having tried more. To study this point further, one extra bar was made of iron No. 881, careful profile measurements made with "feeler" gages, and the bar subjected to repetitions of a calculated bending stress of 45,000 lb. per sq. in. The change in deflection and the corresponding "set" are as follows:

Loading	Deflection, in.	Set, in.
1	0.265	0.015
2	0.238	.....
3	0.235	0.019
12	0.24	0.008[a]
60	0.238	0.023
240	0.24	0.025
373	0.23	0.022

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[a] The bar was loaded three times in the opposite direction after the third loading, and the set at the twelfth loading shows that there is some effect still left.

Errors in the deflection readings and the profile readings must be allowed for, but it appears certain that the bar has shown full elasticity for the last 313 loadings—one of which (No. 367) was sustained for three hours with no measurable creep on the load-deflection diagram and no drop of the beam. Experiments are being continued on this bar, but the work already done indicates the need of a careful investigation along these lines with accurate apparatus, and with due regard to the fatigue limit in repeated bending. This may well show a great superiority over the fatigue limit in reversed bending, for such bending continually repeats the plastic flow, whereas this does not continue in the ordinary service demanded of the material. If the fatigue limit is the true elastic limit, which seems probable, then it may well be that cast iron is far better than is commonly supposed for repeated stress acting in one direction.

When applied under clearly defined conditions, the modulus of elasticity is one of the most useful indications of the quality of cast iron. It was shown by the author[1] that it is very closely related to the analysis when cooling conditions are uniform and therefore is a good index of the "coulability," or casting properties, of the iron. "Coulability" increases as the modulus of elasticity decreases until the eutectic is passed. But inasmuch as the strength rapidly falls off beyond this point, the specification for strength would be an effective guarantee against too low a modulus of elasticity. The resilience, or the area below the stress-strain diagram and thus a function of both strength and modulus of elasticity, therefore deserves a place in our specifications for cast iron.

The modulus of elasticity has been used in British standard specifications for piston rings, by the United States Government in its master specification for centrifugally-cast cast-iron pipe, and is recognized in some municipal specifications for cast-iron pipe by a minimum requirement for deflection at some given load point—thus placing a maximum on the modulus of elasticity at that point.

[For Discussion on Properties of Cast Iron, see page 142.—Ed.]

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[1] J. T. MacKenzie, "Influence of Phosphorus on Cast Iron," *Transactions*, Am. Foundrymen's Assn., p. 988 (1926).

THE FATIGUE PROPERTIES OF CAST IRON

By J. B. Kommers[1]

Synopsis

The paper reports the results obtained in testing four lots of cast iron at the University of Illinois and ten different cast irons at the University of Wisconsin.

The Illinois tests included tension, compression, Charpy impact, Brinell hardness, and rotating-beam fatigue tests. The fatigue strength of cast iron was markedly increased by oft-repeated stress below the endurance limit. Grooves in cast-iron test-specimens reduced the endurance limit only a small amount. Fatigue, tension, and Brinell hardness tests were made at elevated temperatures; there was no great reduction in these values up to a temperature of about 800° F.

Tests on range of stress showed that the endurance limit for stresses from zero to a maximum tension were 1.48 times the endurance limit for completely reversed flexural stress. Formulas are given by means of which the maximum unit stress for various ratios of minimum to maximum stress may be computed approximately.

The Wisconsin tests included tension, compression, transverse, Russell impact, Rockwell and Brinell hardness, and rotating-beam fatigue tests. The materials ranged from low-strength to high-strength cast irons. The ratio of endurance limit to tensile strength showed an average value of 0.49; and the ratio of endurance limit to modulus of rupture showed an average value of 0.26

The results described in this paper are taken from the work done at the University of Illinois by H. F. Moore[2] and his associates, and that done at the University of Wisconsin by the present author.[3] Aside from these results, so far as the author is aware, there have been published only a few isolated tests on the fatigue of cast iron.

University of Illinois Tests

The University of Illinois tests were made on four different lots of cast iron: one from a 6-in. cast-iron pipe made by the centrifugal process, and the other three from hollow cylinders from 1 to 3¼ in. thick.

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[1] Professor of Mechanics, Engineering College, University of Wisconsin, Madison, Wis.

[2] H. F. Moore, S. W. Lyon, and N. P. Inglis, "Tests of the Fatigue Strength of Cast Iron." *Bulletin No. 164*, Engineering Experiment Station, University of Illinois; see also H. F. Moore and S. W. Lyon, "Tests of the Endurance of Gray Cast Iron Under Repeated Stress," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 87 (1927).

[3] J. B. Kommers, "The Static and Fatigue Properties of Some Cast Irons," *Proceedings*, Am. Soc. Testing Mats., Vol. 28. Part II, p. 174 (1928).

Results of Tests:

Table I shows the results of the various static, impact and rotating-beam endurance tests.

Table II shows the ratios of the endurance limit to the several properties reported in Table I.

Table I.—Summary of Results of Tests, University of Illinois.

Cast Iron	Tensile Strength, lb. per sq. in.	Compressive Strength, lb. per sq. in.	Charpy Tests (notched bar), ft. lb.	Brinell Hardness Number	Endurance Limit, lb. per sq. in.
No. 91	26 200	96 000	4.0	162	12 000
No. 92	31 000	111 000	11.8	148	10 500
No. 93A	28 100	94 000	2.7	138	10 000
No. 93B	25 300	85 000	2.2	132	9 000
No. 93C	28 000	87 300	2.5	139	10 000
No. 94A	21 400	82 500	1.6	89	7 000
No. 94B	20 700	83 500	1.6	91	7 200
No. 94C	20 500	82 000	1.5	88	7 800

Table II.—Endurance Limit Ratios.

Cast Iron	Ratio, Endurance Limit to Tensile Strength	Ratio, Endurance Limit to Compressive Strength	Ratio, Endurance Limit to Charpy Tests	Ratio, Endurance Limit to Brinell Hardness
No. 91	0.46	0.125	3 000	74.0
No. 92	0.33	0.095	890	71.0
No. 93A	0.36	0.106	3 710	72.5
No. 93B	0.36	0.106	4 090	68.2
No. 93C	0.36	0.115	4 000	72.0
No 94A	0.33	0.085	4 380	78.6
No. 94B	0.35	0.086	4 500	79.2
No. 94C	0.38	0.095	5 200	89.6
Average	0.37	0.102	3 720	75.6
Mean Variation from the Average, per cent	7.8	10.8	20.6	5.1

These results indicate that the most consistent relations are the ratios of the endurance limit to the tensile strength and to the Brinell hardness number.

Understressing Tests:

Tests of metals and also of concrete have shown that when a material is stressed at or just below the endurance limit for a considerable number of cycles of stress, there is produced a strengthening effect such that the new endurance limit of the material is higher than it was before such understressing.

The Illinois tests included understressing tests on cast irons Nos. 93 and 94. It was found that the new endurance limit after understressing exceeded the original endurance limit by amounts varying between 16 and 43 per cent.

The author is at present carrying out a series of tests on the effect of understressing. The results on a low-strength cast iron indicate that after about 20,000,000 cycles of understressing the maximum strengthening effect has been attained, and also that the most effective stress to use for strengthening is one which is as close as possible to the endurance limit of the virgin material.

Tests on concrete have shown that even for unit stresses under which the material does not ultimately fail there is a permanent deformation during the first several hundred thousand cycles of stress, and that the material has the capacity to adjust itself to the stress cycle. The permanent deformation with continued repetitions finally becomes constant and does not increase, and the material then seems to be fully elastic up to and slightly above the value of the unit stress to which it has been subjected. It seems most probable that in cast iron there is also some permanent deformation at first, and that the material becomes adjusted to the applied cycle of stress so long as that stress is less than the endurance limit.

Effect of Holes and Grooves on Fatigue Strength:

The Illinois tests included experiments on the effect of holes and grooves in lowering the endurance limit. Tests on various metals have shown that the stress intensification due to abrupt changes of cross-section, as given by the mathematical theory of elasticity, are greater in percentage than the actual reduction of endurance limit as obtained in fatigue tests.

Specimens of cast iron No. 93 tested with a radial hole 0.055 in. in diameter showed a theorectical reduction of endurance limit of 67 per cent, while the actual reduction in the fatigue tests was only 13 per cent. Specimens of cast iron No. 92 were tested with two different types of filleted grooves. The theoretical reduction of endurance limit to be expected was 74 per cent, while the actual reduction in the fatigue tests was only zero and 8 per cent, respectively.

Tests which the author has made on a low-strength cast iron in which a square notch had been cut on the specimen, showed an endurance limit for the notched specimen about 500 lb. per sq. in. greater than that for a standard specimen without a notch.

Carbon flakes in cast iron may be looked upon as notches which evidently occur on the surface of the specimen and also internally.

The above tests on square-notched specimens indicate that the notching due to the carbon flakes was more serious than the square notch cut on the specimen. The shoulders of the square notch apparently acted to strengthen the metal at the corner of the notch, with the result that the endurance limit was slightly increased.

The mathematical theory of elasticity assumes that the material is stressed within the elastic limit. A material like cast iron has a poorly defined elastic limit, and with notched and grooved specimens it is certain that permanent deformations occur. The tests indicate that so long as these deformations do not exceed a certain amount the material can adjust itself to the deformations, and after a sufficient number of cycles of stress the permanent deformations cease to increase. The material will then be perfectly elastic within the range of the applied stress.

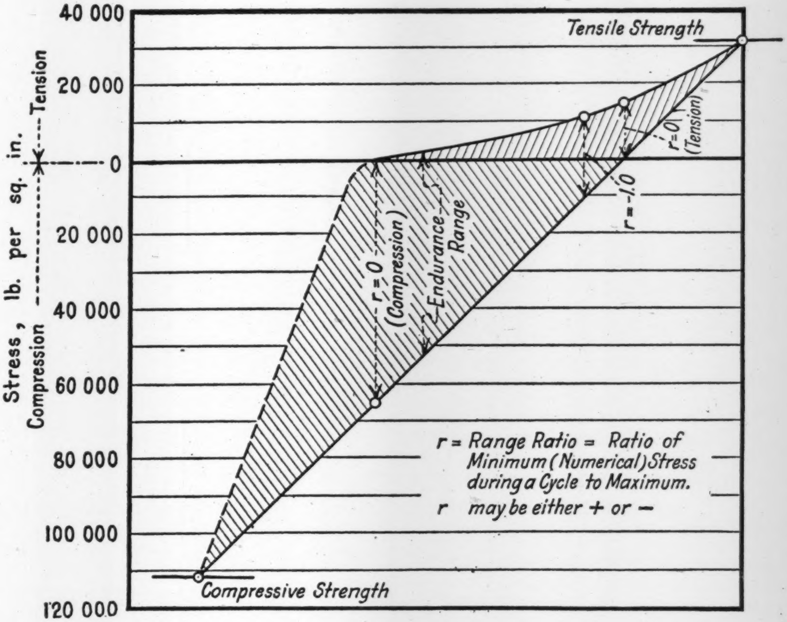

FIG. 1.—Diagram for Varying Endurance Limits of Cast Iron with Varying Range of Stress.

Test at Elevated Temperatures:

The Illinois tests included experiments on cast iron No. 92 at various elevated temperatures, ranging up to a maximum of 1400° F. The tests indicated that the tensile strength, Brinell hardness and endurance limit decreased very little up to about 800° F., but that beyond this temperature there was a decrease, which was much more rapid for the tensile strength and Brinell hardness than for the endurance limit.

In this connection may be mentioned one series of tests made at the University of Wisconsin on a cast iron which had 0.63 per cent of nickel and 0.22 per cent of chromium. This material was heated to 950° F., held for three hours and allowed to cool. It was found that all the static and fatigue results were practically the same as for the material which had not been subjected to such heat treatment. These results indicate that heating the material to 950° F. and cooling to room temperature had not resulted in a measurable change in the strength properties.

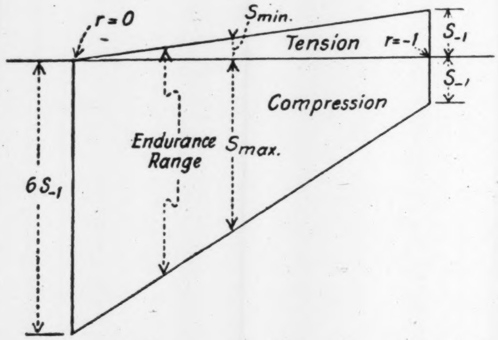

FIG. 2.—Portion of Diagram Shown in Fig. 1, from Maximum Compressive Endurance Range to Maximum Tensile Endurance Range.

Tests on Range of Stress:

Figure 1 shows the results obtained when fatigue tests were made using various combinations of tension and compression in the cycle of applied stress. When the unit stress was entirely compressive from zero to a maximum ($r = 0$), the endurance range was 65,000 lb. per sq. in., representing the endurance limit in compression. When a small amount of tension was introduced into the cycle the endurance range was at once reduced. For example, at 6400 lb. per sq. in. tensile stress in the cycle the endurance range becomes 38,400 lb. per sq. in. For completely reversed stress, that is tensile stress equal to compressive stress ($r = -1$), the endurance range is reduced to 21,000 lb. per sq. in., the endurance limit being 10,500 lb. per sq. in. Finally, when the unit stress was entirely tensile from zero to a maximum ($r = 0$), the endurance range was smallest of all, being only 15,500 lb. per sq. in., corresponding to the endurance limit in tension.

TABLE III.—SUMMARY OF RESULTS OF TESTS, UNIVERSITY OF WISCONSIN.

	Tensile Strength		Compression Test		Hardness Test			Transverse Test			Impact	Fatigue
	Small Specimen, lb. per sq. in.	Large Specimen, lb. per sq. in.	Unit Stress at First Bend, lb. per sq. in.	Unit Stress at Ultimate, lb. per sq. in.	Brinell Hardness Number	Rockwell Hardness Number, "B" Scale		Maximum Load, lb.	Maximum Deflection, in.	Modulus of Rupture, lb. per sq. in.	Energy of Rupture, in lb. per cu. in.	Endurance Limit, lb. per sq. in.
						At Center	At Mid-radius					
Series A.	34 900	28 300	77 200	109 200	197.0	90.0	95.0	5030	0.162	78 700	77.3[a]	
	34 600	39 300	76 200	107 900	201.0	87.5	93.0	4630	0.129	72 700	44.2	
	37 500	37 600	70 000	109 800	203.0	89.0	94.0	4750	0.159	74 100	38.9	
	.....	.....	.....	......	203.0	85.0	93.0	4730	0.156	74 600	34.4	
	.....	.....	.....	......	207.0	90.0	93.0	4430	0.140	69 200	31.0	
Average...	35 700	35 100	74 500	109 000	202.0	88.3	93.6	4714	0.149	73 900	37.1	19 000
Series B.	38 700	42 000	73 600	138 700	207.0	96.5	98.3	4280	0.118	67 000	22.5	
	40 000	39 600	78 100	138 000	207.0	98.0	99.0	4110	0.112	64 300	16.4	
	39 800	37 400	81 500	136 100	212.0	95.8	98.0	4040	0.111	63 300	19.8	
	.....	.....	.....	......	201.0	93.0	98.0	4200	0.111	65 800	19.2	
	.....	.....	.....	......	209.5	98.5	99.0	4220	0.115	66 300	27.8	
Average...	39 500	39 700	77 700	137 600	207.3	96.3	98.5	4170	0.113	65 300	21.1	19 000
Series C.	27 500	31 400	81 200	117 800	197.0	90.5	97.5	3880	0.124	60 800	17.2	
	32 700	32 600	75 700	114 500	192.0	90.0	98.0	3560	0.097	55 800	17.0	
	30 800	33 000	80 400	120 400	197.0	88.5	96.0	3720	0.111	58 400	17.0	
	.....	.....	.....	......	189.5	87.5	96.0	3720	0.113	58 300	22.5	
	.....	.....	.....	......	192.0	92.0	95.0	3640	0.112	56 800	17.5	
Average...	30 300	32 300	79 100	117 600	193.5	89.7	96.5	3704	0.111	58 000	18.2	15 000
Series D.	33 100	31 700	83 300	123 800	194.5	93.0	94.0	4030	0.127	63 200	23.6	
	33 000	28 800	83 100	121 600	192.0	88.0	96.0	3740	0.100	56 200	22.9	
	29 600	30 600	81 500	123 000	192.0	93.0	97.0	3580	0.120	58 300	21.6	
	.....	.....	.....	......	187.0	88.0	97.0	3800	0.111	59 300	21.6	
	.....	.....	.....	......	197.0	84.0	96.0	3960	0.125	62 000	18.6	
Average...	31 900	30 400	82 600	122 800	192.5	89.2	96.0	3822	0.117	59 800	21.7	12 000
Series E.	32 500	30 600	77 700	116 900	185.8	91.5	97.0	3520	0.122	55 300	24.4	
	32 100	31 500	85 600	118 800	187.0	94.5	96.0	3760	0.126	58 700	23.3	
	31 200	31 100	74 300	115 400	179.0	93.5	96.0	3960	0.140	62 300	26.5	
	.....	.....	.....	......	183.0	96.0	96.0	4000	0.145	62 700	20.6	
	.....	.....	.....	......	187.0	92.5	95.0	3900	0.138	61 300	22.2	
Average...	31 900	31 100	79 200	117 000	184.4	93.6	96.0	3830	0.134	60 100	23.4	13 400
Series F.	38 600	35 700	82 700	133 200	209.5	95.5	97.5	4600	0.153	72 500	30.6	
	35 400	36 400	82 800	129 100	214.5	97.5	97.5	4260	0.133	66 800	25.1	
	37 200	34 600	80 900	133 700	209.5	95.0	97.5	4670	0.148	73 500	27.8	
	.....	.....	.....	......	207.0	97.0	98.0	4080	0.121	64 000	17.3	
	.....	.....	.....	......	207.0	94.0	97.5	4120	0.112	64 400	23.8	
Average...	37 100	35 600	82 100	132 000	209.5	95.8	97.6	4346	0.133	68 200	24.9	19 500
Series G.	36 700	33 400	76 300	128 700	194.5	88.0	93.0	4360	0.140	68 600	18.4	
	37 300	33 000	77 500	135 900	198.5	95.0	96.5	4660	0.171	73 200	30.7	
	32 900	34 800	78 800	128 200	197.0	92.0	96.5	3760	0.111	59 000	34.9	
	.....	.....	.....	......	197.0	96.0	96.0	4620	0.168	72 300	23.6	
	.....	.....	.....	......	201.0	93.0	96.0	4490	0.168	70 200	26.4	
Average...	35 600	33 700	77 500	130 900	197.6	92.8	95.6	4378	0.152	68 700	26.8	19 100
Series H.	55 500	51 500	83 300	158 500	241.0	100.5	101.0	5720	0.180	89 800	45.1	
	43 200	48 400	83 600	145 000	248.0	100.0	100.0	5460	0.162	85 500	41.6	
	54 200	44 300	81 100	165 300	235.0	99.0	101.0	5530	0.185	86 800	48.1	
	.....	.....	.....	......	237.0	100.0	100.0	5660	0.190	88 700	43.6	
	.....	.....	.....	......	233.0	102.0	101.0	5770	0.189	90 400	41.5	
Average...	51 000	48 100	82 700	156 300	238.8	100.3	100.6	5628	0.181	88 200	44.0	22 000

[a] Not included in determining the average.

NOTE.—The loads in the transverse test have been reduced to a common diameter of 1.25 in.

TABLE III.—SUMMARY OF RESULTS OF TESTS.—*Continued.*

	Tensile Strength		Compression Test		Hardness Test			Transverse Test			Impact	Fatigue
	Small Specimen, lb. per sq. in.	Large Specimen, lb. per sq. in.	Unit Stress at First Bend, lb. per sq. in.	Unit Stress at Ultimate, lb. per sq. in.	Brinell Hardness Number	Rockwell Hardness Number, "B" Scale		Maximum Load, lb.	Maximum Deflection, in.	Modulus of Rupture, lb. per sq. in.	Energy of Rupture, lb. per cu. in.	Endurance Limit, lb. per sq in.
						At Center	At Mid-radius					
Series J.	23 600	23 600	74 000	86 000	179.0	86.5	88.0	3260	0.113	51 100	15.9	
	26 000	24 700	73 000	85 000	174.0	87.0	89.0	3690	0.141	57 700	16.7	
	20 000	26 500	74 000	84 000	179.0	90.0	88.5	3520	0.127	55 100	20.0	
	.....	.....	.....	......	174.0	86.5	89.5	3200	0.102	50 100	17.7	
	.....	.....	.....	......	152.0	84.0	85.5	3580	0.134	56 100	12.4	
Average...	23 200	24 900	73 700	85 000	171.6	86.8	88.1	3450	0.123	54 000	16.5	11 800
Series K.	41 900	47 600	79 200	152 800	241.0	92.5	99.0	4830	0.132	75 500	36.0	
	40 700	52 300	75 600	152 600	229.0	95.5	99.0	5390	0.125	84 300	28.6	
	45 100	51 000	82 000	158 800	235.0	96.5	98.5	4840	0.121	76 000	....	
	.....	.....	.....	......	244.5	98.5	99.0	4340	0.100	67 400	.....	
	.....	.....	.....	......	235.0	97.5	98.5	4780	0.119	74 800	....	
Average...	42 500	50 300	78 900	154 700	236.9	96.1	98.8	4836	0.119	75 600	32.3	24 100

NOTE.—The loads in the transverse test have been reduced to a common diameter of 1.25 in.

The endurance range, therefore, in these tests varied from 65,000 lb. per sq. in. when the stress was entirely compressive, to 15,500 lb. per sq. in. when the stress was entirely tensile. It is evident, therefore, that, for cast iron, tensile stress is especially effective in producing fatigue failure.

If S_{-1} is the endurance limit for completely reversed stress, r is the ratio of the minimum to the maximum unit stress, and $S_{\max}$ is the maximum unit stress for any cycle, then the following equation[1] may be used for that portion of Fig. 1 in which the tensile unit stress is numerically the maximum:

$$S_{\max} = \frac{3\,S_{-1}}{2 - r} \qquad \text{(1)}$$

For the portion of Fig. 1 in which the compressive stress is numerically the maximum $S_{\max}$ may be obtained from the following equation:

$$S_{\max} = \frac{6\,S_{-1}}{1 - 5r} \qquad \text{(2)}$$

Equation 2 covers the portion of the diagram shown in Fig. 2, which is the same as that portion of Fig. 1 in which the compressive unit stress is numerically the maximum, except that the upper limit of tensile stress has been approximated by a straight instead of a curved line. Taking the value of S_{-1} as 10,500, the tests show that when $r = 0$ the endurance limit is practically $6\ S_{-1}$. When $r = -0.2$ the equation gives a value of 31,500 lb. per sq. in. for $S_{\max}$. The experiments gave a value of 32,000 lb. per sq. in.

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[1] See "Fatigue of Metals" by H. F. Moore and J. B. Kommers, p. 185., McGraw-Hill Book Co., 1927.

It should be noted that in both Eqs. 1 and 2, r is minus when the cycle of stress is partly tensile and partly compressive.

UNIVERSITY OF WISCONSIN TESTS

The experiments carried out at the University of Wisconsin involved tests on ten different cast irons, covering the range from low-strength to high-strength materials. The specimens were all prepared from standard arbitration bars as specified by the American Society for Testing Materials.[1]

Table III is a summary of the results obtained from the various static, impact, and fatigue tests.[2]

TABLE IV.—CORRELATION BETWEEN ENDURANCE LIMIT AND OTHER TEST RESULTS, UNIVERSITY OF WISCONSIN.

	Ratio, Endurance Limit to Tensile Strength	Ratio, Endurance Limit to Compressive Strength	Ratio, Endurance Limit to Rockwell Hardness	Ratio, Endurance Limit to Brinell Hardness	Ratio, Endurance Limit to Modulus of Rupture	Ratio, Endurance Limit to Energy of Rupture
Series A	0.53	0.174	216	94.0	0.26	512
Series B	0.48	0.138	197	91.8	0.29	900
Series C	0.50	0.128	168	77.6	0.26	825
Series D	0.38	0.98	135	62.3	0.20	553
Series E	0.42	0.115	143	72.7	0.22	573
Series F	0.53	0.148	204	93.2	0.29	783
Series G	0.54	0.146	206	96.8	0.28	713
Series H	0.43	0.141	219	92.2	0.25	500
Series J	0.51	0.139	136	68.8	0.22	715
Series K	0.57	0.156	251	101.8	0.32	746
Average	0.49	0.138	187	85.1	0.26	682
Mean Variation from the Average, per cent	10.0	10.8	18.0	13.9	11.2	17.3

Table IV gives the ratio of the endurance limit to the various other results obtained.[2] The ratios of endurance limit to tensile strength, to Brinell hardness, and to modulus of rupture seem to be the most consistent. The ratio of endurance limit to tensile strength averaged about 0.50. The ratio of endurance limit to modulus of rupture averaged about 0.25.

It will be noted that the ratios of endurance limit to tensile strength, compressive strength, and Brinell hardness in Table IV are somewhat higher than those given in Table II for the Illinois results.

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[1] See Standard Specifications for Gray-Iron Castings (A.S.T.M. Designation: A 48 – 18), 1927 Book of A.S.T.M. Standards, Part I, p. 441.

[2] Reprinted from paper by J. B. Kommers, "The Static and Fatigue Properties of Some Cast Irons," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 174 (1928).

It is possible that the higher endurance limits obtained in the Wisconsin tests may be due to the fact that special precautions were taken to prevent vibrations from decreasing the endurance limit. Sometimes a specimen did not run true, and the resulting vibrations have been counteracted in the Wisconsin tests by mounting the frame of the fatigue machine on four coiled springs, and also by hanging the applied load from another coiled spring. These springs prevent the vibrations of the specimen from being transmitted to the load which is being applied to the specimen, thus preventing acceleration effects which would occur if the weights vibrated.

H. F. Moore has made the following comment on the above statement: "This is in harmony with general observations during the Illinois tests. In previous tests of rolled steel it had been observed that there could be a considerable vibration of the specimen with very little effect on the endurance limit. With cast iron it was found that test results from specimens showing appreciable vibration to the unaided eye gave low and inconsistent values for strength. The additional precautions taken at Wisconsin to avoid the effects of vibration may well account for the increased endurance ratio."

[For Discussion on Fatigue Properties of Cast Iron, see page 163.—ED.]

IMPACT TESTING OF CAST IRON

By H. Bornstein[1]

Synopsis

The impact or shock testing of cast iron has been studied by many investigators. No standard method of test has been developed, although a number of methods have been proposed, the results of which are not in general agreement. There is need for a standard method of test.

Cast iron is not a ductile material. An examination of its structure shows why it lacks ductility. The size, shape, quantity and distribution of the graphite affect the static and impact values. While there is some disagreement between investigators, it is the consensus of opinion that irons high in static strength are also high in impact value. In many cases, the shock or impact test is of greater value in predicting results in service than are static tests, particularly where the casting is to be subjected to shock.

There is no standard test or method for measuring the resistance of cast iron to impact, although several methods have been proposed. One method of determining impact value is to measure the energy absorbed in the breaking of the test specimen. This is done in such apparatus as the Izod, Charpy, and Olsen impact testing machines. These machines consist essentially of three principal parts:[2] a moving mass having a known kinetic energy at the instant the blow is struck, an anvil on which the specimen is mounted, and a device for measuring the kinetic energy in the moving mass after the specimen has been ruptured.

Another type of impact test is to support the test specimen in a horizontal position on two supports and then drop a hammer on the specimen midway between supports. The hammer is dropped from successively increasing heights until failure occurs.

In another type of machine, developed by the British Cast Iron Research Association,[3] a hammer of known weight is dropped from a known height onto the test specimen which is turned through 180 deg. between successive blows. The number of blows required to break the bar is recorded.

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[1] Director of Laboratories, Deere and Co., Moline, Ill.

[2] F. N. Menefee and A. E. White, "A Study of Centrifugally-Cast Pipe (Metal-Mold Process) *versus* Sand-Cast Pipe," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 221 (1928).

[3] "Impact Testing of High Duty Cast Iron," *Foundry Trade Journal*, Vol. 35, p. 98 (1927).

There has been no agreement as to which of these types of test gives information of most value. Furthermore, the results have not been comparative.

Cast iron is not a ductile material and, consequently, its shock value, compared with steel, is relatively low. Ductility is usually measured by the elongation and reduction of area in the tension test and cast iron has scarcely any measurable amount of elongation or reduction of area.

There is considerable variation in the impact values of different cast irons. This is probably due to differences in structure, caused by differences in composition, cooling rates, etc. Cast iron is low in shock value because of its structural formation, having graphite, ferrite, pearlite, cementite, etc. The size and shape of the graphite are large factors in determining both the static and impact strength. The graphite flakes can be considered as so many voids, in so far as static strength is concerned, and in impact tests they give a notch effect. Malleable cast iron can be considered as a cast iron in which the graphite particles are very small and rounded and the matrix is ferrite. Malleable cast iron is much stronger than ordinary cast iron, very ductile, and quite resistant to shock. This would indicate that for both static and impact strength it is desirable to keep the carbon content low and have the graphite in small particles.

Ferrite is more ductile than pearlite. Consequently, a cast iron which has a ferrite matrix with small flakes of graphite should be relatively high in shock value, while an iron with small graphite particles but with pearlite matrix would give highest results in static tests. Unfortunately, so far as impact values are concerned, in ordinary cast irons the larger quantities and size of graphite go with the ferrite matrix. Of the commercial irons, the so-called pearlitic irons give the best results in static tests and also give high values in shock resistance. In the author's laboratory, shock tests were made to compare "regular" cast iron with high-strength iron. Both irons were made in the cupola. For the "regular" iron, the charge consisted of 40 per cent of pig iron, 10 per cent of steel scrap, and 50 per cent of returns and gray iron scrap. For the high-strength iron, the charge consisted of steel scrap (rails) plus sufficient ferro-manganese to give 0.75 per cent of manganese in the iron and sufficient ferrosilicon to give about 1.5 per cent of silicon in the iron. Ferrosilicon and nickel shot were added in the ladle to give a silicon content of 2.25 per cent and a nickel content of 1.25 per cent. Following are typical analyses and results of static tests on the two irons:

	"REGULAR" IRON	HIGH-STRENGTH IRON
Silicon, per cent	2.05	2.25
Sulfur, per cent	0.075	0.075
Manganese, per cent	0.60	0.75
Phosphorus, per cent	0.35	0.080
Combined Carbon, per cent	0.50	0.55
Total Carbon, per cent	3.30	2.90
Nickel, per cent	....	1.25
Tensile Strength, lb. per sq. in	33 500	61 600
Transverse Test, Arbitration Bar:		
Breaking Load, lb	4 100	6 050
Deflection, in	0.120	0.175

The transverse tests were made on the 1¼-in. diameter arbitration bar and the tension tests were made on 0.8-in. diameter specimens turned from broken halves of the arbitration bars.

Impact tests were made on broken halves of the arbitration bars. The test specimens were placed horizontally on supports 4 in. apart and a 100-lb. hammer was dropped midway between the supports. The first drop was from a height of 2 in.; successive drops were made at increments of 2 in. to a height of 10 in., and then with 1-in. increments above 10 in. With the "regular" iron, out of 30 tests, 15 test bars broke at a drop of 6 in., 12 at 8 in. and 3 at 10 in., giving an average height of drop of 7.2 in. Out of 30 tests with the high-strength iron, 2 broke at 11 in., 2 at 12 in., 1 at 13 in., 3 at 14 in., 5 at 15 in., 7 at 17 in., 4 at 18 in., and 6 at 20 in. This gave an average height of drop of 16.2 in.

We have found the drop test to be valuable in making tests on castings that are to be subjected to shock in service for the results of drop tests give much closer information as to what to expect in service than do static tests.

WORK OF OTHER INVESTIGATORS

The subject of impact testing of cast iron has received study from many investigators. Keep[1] goes into the subject of impact testing to a considerable extent. He describes an impact testing machine using a hammer swinging on a vertical arm 6 ft. long. The weight of the hammer can be varied between 25 and 100 lb. The height of drop is gradually increased until failure occurs. Blows should be begun with the same drop at all times, which should be less than the lowest possible breaking drop, and then each drop should be increased by ⅛ in. until fracture occurs. Keep prefers this swinging hammer impact test and states it appears to be the best mode of application of impact for ordinary cast-iron test bars. He also describes a conventional drop test.

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[1] W. J. Keep, "Cast Iron: A Record of Original Research," 1909, John Wiley and Sons.

Keep brings out the fact discovered by Outerbridge[1] that test bars tumbled in a tumbling barrel are always stronger than companion bars not tumbled. Keep made a large number of tests and came to the conclusion that the great increase in strength is caused, not so much by the grains moving on each other and readjusting themselves, but by the condensing of the grains by pounding, and by smoothing, thus removing notches which would induce fracture.

Portevin[2] gives a review of physical tests carried out during the war on semi-steel castings for shells. He reports that shock or impact tests gave inconsistent results.

O. Bauer[3] describes results of tests on ordinary cast iron, cylinder iron and pearlitic cast iron. He reports that repeated impact tests give considerably higher results on the pearlitic cast iron than on the other two irons.

F. J. Cook[4] describes an impact or shock test for cast iron and states that the test is becoming general in Europe for cast irons of the highest physical properties, more particularly for Diesel and large gas-engine pistons and cylinder liners. This is carried out by testing a bar cast 40 mm. (1.58 in.) square supported on knife edges 160 mm. (6.3 in.) apart by dropping onto it a weight of 12 kg. (26.46 lb.) from varying heights. Attached to the weight in such a way as to strike the bar in the center, parallel to the supporting knife edges, is fixed another knife edge. The faces of all the knife edges are rounded to $\frac{1}{16}$-in. radius. In carrying out the test, a start is made with a drop of the weight from a height of 30 cm. (11.81 in.), increasing the height of drop by increments of 5 cm. (1.97 in.) until the sample breaks, the height at which failure occurs being taken as the test figure. It is stated that a result of 55 cm. (21.65 in.) is considered none too high for the class of work named, although it is quite a severe test. The maximum test figure obtained was 88 cm. (34.65 in.).

Kommers[5] made tests on ten different irons from eight manufacturers making a wide range of products. These tests included static, fatigue and impact tests. "The impact tests were made on a Russell impact machine, using a 12-in. span. The Russell machine consists essentially of a slab of steel weighing 185 lb., which can rotate as a pendulum about an axis at one end. By means of suitable dials, the drop of the center of gravity of the slab before striking the specimen and the rise after breaking the specimen may be measured and thus the energy of rupture calculated." An examination of Kommers' data shows that there was no definite relationship between impact values and results of tension, compression, transverse and fatigue tests. However, the highest results for "energy of rupture" were secured with the irons giving highest results for breaking loads in tension and transverse tests. Similarly the irons giving low results in impact test gave low results in other tests.

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[1] A. E. Outerbridge, *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 26, p. 176 (1896).

[2] A. Portevin, *Revue de Metallurgie*, Vol. 18, p. 761 (1921).

[3] O. Bauer, "Manufacture, Tensile Properties and Applications of Pearlitic Cast Iron," *Stahl und Eisen*, Vol. 43, p. 553 (1923).

[4] F. J. Cook, "American *versus* British Gray Cast Iron," *Transactions*, Am. Foundrymen's Assn. Vol. 30, p. 132 (1922).

[5] J. B. Kommers, "The Static and Fatigue Properties of Some Cast Irons," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 174 (1928).

The results obtained by Kommers are of interest because of the results reported by Pearce.[1] He stated that in many cases high-tensile-strength irons were secured at the expense of the little elongation that good cast iron possesses, and that an extra-high-tensile-strength iron was frequently extremely brittle. Pearce discussed the Izod impact test and the Wöhler fatigue test, in which a round bar is held horizontally by one end in a lathe chuck and rotated, a load being suspended at the other end. He concluded that something between the sudden shock, single-blow test and the Wöhler fatigue test was required.

He then described a commercial impact testing machine which was being used by the Cast Iron Research Association for testing of cast iron. The test consists of dropping a hammer of known weight from a known height, the hammer being operated by means of a cam. The test bar is about $\frac{5}{8}$ in. in diameter with a groove in the center. It is fitted into the machine horizontally and rotated, being shaped at each end to ensure proper holding and control of rotation. The average number of blows is from 60 to 90 per minute, and between each blow the bar is turned through 180 deg. so that no two successive blows fall on the same spot. The number of blows required to break a bar is recorded automatically. There are two heights from which the hammer can be made to fall, and there are two hammers, each of different weight. In addition, an extra weight can be added to one hammer, if desired, so that there would be six different intensities of blow to suit the particular metal being tested. There is also an approximate means of converting the figures obtained with one type of hammer into the figures obtained with another type, so that comparisons could be made. A number of results of tests on various irons were then given, the figures being averages of a number of tests on similar irons in each case. In some cases there was a wide variation in the number of blows needed to break test bars made under exactly the same conditions and of the same composition, pointing to internal strain and lack of homogeneity.

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[1] J. G. Pearce, "Impact Testing of High Duty Cast Iron," *Foundry Trade Journal*, Vol. 35, p. 98 (1927).

Pearce did not claim that the results of this test would enable one to say whether an iron was good or poor but, on the evidence so far obtained, it showed that an iron which behaved well under this test was likely to behave well under service conditions.

Talbot and Richart[1] made impact tests on cast-iron pipe. The pipe was placed on two supports 10 ft. apart, filled with water, kept under water pressure (generally about 65 lb. per sq. in.) and subjected to blows of a 50-lb. hammer dropped midway between supports from increasing heights until a crack occurred. The results of this test showed that the resistance to impact varied with the square of the wall thickness. Menefee and White[2] ran impact tests on specimens cut from cast-iron pipe. The Izod, Olsen and "Baby" Olsen impact testing machines were used. It was found that annealing measurably increased the impact resistance of centrifugally-cast pipe.

[For Discussion on Impact Testing of Cast Iron, see page 165. —ED.]

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[1] A. N. Talbot and F. E. Richart, "A Study of the Relation Between the Properties of Cast-Iron Pipe," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 185 (1926).

[2] F. N. Menefee and A. E. White, *loc. cit.*

WEAR TESTING OF CAST IRON

BY A. L. BOEGEHOLD[1]

SYNOPSIS

The object of this paper is to review the work that has been done on wear testing in general and in particular the work done on wear testing of cast iron, and to explain some of the difficulties encountered in obtaining satisfactory results from wear tests of this material.

The various investigations which have been conducted on the wear resistance of cast iron are touched on briefly, and the outstanding conclusions reached and the opinions as to the causes for resistance to wear of cast iron are compared. There is no good agreement between various investigators as to the method of testing for wear of cast iron nor in their opinions regarding the influence of various elements in cast iron upon resistance to wear.

Regarding the wear testing of cast iron for automobile engine cylinder blocks, the author describes a method of test used by him and explains difficulties encountered in trying to conduct the wear test with a lubricant present. A brief description is given of a wear test conducted on an automobile engine constructed of individual cylinders of four different kinds of cast iron, which after running 20,000 miles, demonstrated that the kind of cylinder iron used had no influence upon the result because of good lubrication between the piston and the cylinder.

The conclusions reached are:

1. A universal wear test is undesirable.
2. Laboratory wear tests in which service conditions are imitated give valuable data.
3. Wear testing with lubricants present is difficult.
4. Grit and lack of lubrication are the largest factors in cylinder wear.
5. Investigators of cast-iron wear should have comprehensive knowledge of cast iron.

In an article entitled "The Wear of Metals and Its Determination," the editor of ***Engineering***[2] cites the statement, "It is much to be regretted that there is still no laboratory or workshop test that provides a reliable result concerning resistance to wear," from a paper entitled "Chromium Steel Rails" submitted at the 1928 meeting of The Iron and Steel Institute by Dr. T. Swinden and Mr. P.H. Johnson. The editor also states that most authorities agreed that there was no general wear test that could be related to all kinds of wear.

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[1] Metallurgist, General Motors Research Laboratories, Detroit, Mich.

[2] *Engineering*, Vol. CXXVI, No. 3267, August 24, 1928, p. 237.

Because of the variety of ways in which wear takes place, any proposal to test materials for wear immediately makes necessary a more explicit definition of the kind of wear. A list of some of the more common methods by which metallic parts are destroyed by wear is given in the following table:

Type of Wear	Example
Unlubricated sliding surface contact	Brake shoe on car wheels
Lubricated sliding surface contact	Plain bearings Piston against cylinder
Rolling unlubricated	Car wheels on tracks
Rolling lubricated	Roller and ball bearings
Crushing non-metallic abrasives	Rock crushing jaws
Abrasion by non-metallic dry	Steam shovels
Abrasion by non-metallic wet	Pumps handling wet sand
Crushing non-metallic abrasives wet	Wet grinding ore
Soft material against steel lubricated where soft material is continually renewed	Copper wire drawing
Soft material against steel unlubricated where soft material is continually renewed	Flyers used in silk spinning

The information now available as the result of investigations of these various kinds of wear is sufficient to convince the author that there can be no universal wear testing machine or wear test which will give results to indicate how a metal will behave under any set of circumstances.

Whereas the word wear refers to many kinds of mechanical or physical disintegration of metallic surfaces, corrosion is a word that applies to a large number of different chemical processes which are similar, only in that there is caused a wasting away of the metal. Nobody would think of trying to develop a universal corrosion test which would tell how a metal would stand up under any kind of corrosion, yet that is what has been attempted in connection with wear. It seems so self-evident as to be hardly necessary to remark that to obtain test results that mean anything about the performance of a metal in service, that test must be conducted under conditions which duplicate as nearly as possible the conditions existing in the field. That means a special method of wear testing for each type of wear just as special tests are conducted for each type of corrosion.

Some Types of Wear Tests of Metals

Attempts to grade metals as to wear resistance without reference to any particular application of that metal are to be found recorded in the literature. For example: Felix Robin[1] measured the loss of weight of a steel sample after pressing it for three minutes under a known load against a revolving disk of emery paper. It is not clear what relation these results would bear to the wear of any of the steels tested if used in a rock crusher or as a brake shoe or as a railroad frog.

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[1] Felix Robin, "Report on the Wear of Steels," The Iron and Steel Inst., *Carnegie Scholarship Memoirs*, Vol. II, p. 1 (1910).

In the ball mill test, wear is determined by measuring weight lost by balls of the metal to be tested after tumbling them in a drum for a specified length of time. Stanton and Batson[1] produced sliding between a cylindrical specimen and two steel rolls by running the two rolls at different speeds in contact with the cylindrical specimen and applying a load through a third roll mounted on a lever arm. The contact between the rolls being theoretically a line, very light loads will stress the test metal beyond its yield point. As soon as the yield point is reached the area of contact increases and possibly reduces the stress below the yield point. As soon as the deformed region is passed over, the stress is increased again, causing deformation, thus constantly repeating the cycle.

There are other wear tests described in the literature which do not aim to duplicate any specific conditions surrounding the commercial application of metals, but these examples will suffice to illustrate the type.

The development of special wear tests for each type of wear, so that field conditions can be approximated has been the aim of a number of recent investigations. This class of wear testing is illustrated by the Fahrenwald machine described by J. M. Blake,[2] where the service encountered by steels in wet grinding operations is duplicated in a grinding machine where abrasives, pressure, speed, time, and moisture are accurately controlled. The test specimens under load are moved in contact with abrasive material and water in an annular trough.

Hall[3] used a small stone crusher fitted with jaws made of the steel to be tested in determining the best steel to resist the action of stone crushing. He noted that Amsler and Brinell machines did not give results that indicated how a steel would behave in service.

Parker[4] tested tire chains by rubbing them on a grinding wheel under controlled conditions.

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[1] T. E. Stanton and R. G. Batson, "Report of the Hardness Tests Research Committee," *Proceedings*, Inst. Mechanical Engrs., October-December, 1916, p. 677.

[2] J. M. Blake, "Wear Testing of Various Types of Steels," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 341 (1928).

[3] J. H. Hall, "Wearing Tests of Twelve-per-cent Manganese Steel," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 326 (1928).

[4] W. H. Parker, "The Wearing Qualities of Tire Chains," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 332 (1928).

Saniter[1] simulated the action of rolling friction encountered on railroads by rotating a cylindrical specimen and applying a load through a ball bearing whose inner race contacted with the test specimen but was considerably larger in diameter than the test specimen. This produced a rolling motion between the specimen and the inner race of the ball bearing.

French and Herschman[2] built a machine which simulated the conditions attending the inspection of holes in metals by means of plug gages.

Depending upon the test conditions utilized, the Amsler wear testing machine lends itself to making a "universal" wear test or a test for a special kind of wear. French[3] used it under special conditions to duplicate the wear of railroad bearing bronzes.

The work of Robin and others who have attempted to find relations between wear resistance and tensile strength, Brinell hardness, or some other physical property has not been entirely successful, and certainly tests of metals as to wear resistance determined by rubbing against emery paper fails to take into consideration one of the most important phases of wear testing, which is the character of both metals in contact during the wearing process. On the other hand, it is recorded that the results of special wear tests where operating conditions are faithfully simulated, have contained information which indicates the best material to use for the operation in question.

WEAR TESTING OF CAST IRON

A survey of the records on the wear testing of cast iron leaves one with the impression that most of the work on this subject has been rather of a preliminary nature. The discouraging lack of consistent results obtained when working with this material is reason enough for not going very far with the tests. The great number of factors influencing the wearing qualities of cast iron make it very difficult even to get a series of test specimens with which to study the effect of one variable. The possible combinations of two surfaces rubbing in contact is practically infinite.

Various investigators who have studied the wear resistance of cast iron are not in agreement, either in results of their tests or in their opinions as to the influence of the various elements of cast iron upon its resistance to wear. Some of the conclusions reached by these investigators are interesting when compared.

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[1] E. H. Saniter, "A Test for Ascertaining the Relative Wearing Properties of Rail Steel," *Journal*, Iron and Steel Inst., Vol. LXXVIII, No. 3, p. 73.

[2] H. J. French and H. K. Herschman, "Wear of Steels with Particular Reference to Plug Gages," *Transactions*, Am. Soc. Steel Treating, Vol. X, No. 5, p. 683 (1926).

[3] H. J. French, "Wear and Mechanical Tests of Some Railroad Bearing Bronzes," ***Proceedings***, Am. Soc. Testing Mats., Vol. 28, Part II, p. 298 (1928).

Lehman[1] who pressed cast-iron specimens against a rotating drum of iron or steel concluded that:

1. Wear resistance increases with pearlite content;
2. Phosphide eutectic has a harmful influence on wear as it tends to form a grinding substance;
3. Graphite in cast iron acts as a lubricant and diminishes wear;
4. Chemical composition shows no relation to wear; and
5. Brinell hardness shows no relation to wear.

Piwowarsky,[2] however, concluded that phosphorus in the range from 0.1 to 0.75 per cent improved wear resistance, being more helpful than change of hardness produced by superheating.

Lowry[3] also gives data which show that increase of phosphorus improves wear resistance. He considers that wear is governed by carbon forms and raw materials and amount of pearlite but gives no data on this subject.

He describes the glazed surface produced on a soft cast iron in contact with hard cast iron, whereby the hard cast iron wears more in 1000 hours than the soft iron, and that two hard irons together wore faster than either.

Kühnel[4] also gives information on effect of difference in hardness between surfaces in contact. He found in testing locomotive valve rings and valve boxes that ring failure was greatest below Brinell hardness No. 140 and that with a 10 point difference between the hardness of ring and box, the wear is least, being only one-half that when they are both the same hardness. The greater the difference in hardness between ring and box, the greater the wear.

Opposed to Lehman's opinion about graphite, Hurst[5] is quoted in a statement that graphite is not considered to be accountable for the excellent wear resistance of cast iron.

In a series of tests run by Brinell[5] on his abrasion testing machine, he found:

1. Manganese improves the wear resistance of white Swedish charcoal iron which varied from a wear resistance of 287 at 1.47 per cent manganese to 381 at 4.29 per cent manganese.

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[1] O. H. Lehman, *Giesserei Zeitung*, Vol. 23, pp. 597, 623 and 654, November 1, 15 and December 1, 1926.

[2] E. Piwowarsky, *Giesserei Zeitung*, Vol. 43, October 22, 1927, p. 743.

[3] E. J. Lowry, "Cast Iron in Its Relation to the Automotive Industry," *Journal*, Soc. Automotive Engrs., Vol. XX, No. 2, February, 1927, p. 227.

[4] Kühnel, *Giesserei Zeitung*, Vol. 24, October 1, 1927, p. 533.

[5] J. E. Hurst quoted in Bureau Report No. 38 on the Wear of Cast Iron in *Bulletin No. 19*, British Cast Iron Research Assn., January, 1928, p. 10.

2. In a Swedish gray charcoal iron wear resistance was decreased by increasing the silicon content.

3. In a Swedish gray cast iron wear resistance was decreased by increasing the silicon content.

A German committee of iron founders and railroad men[1] found that in general wear decreases with increasing hardness, with many exceptions. Best results were obtained with pearlitic structure.

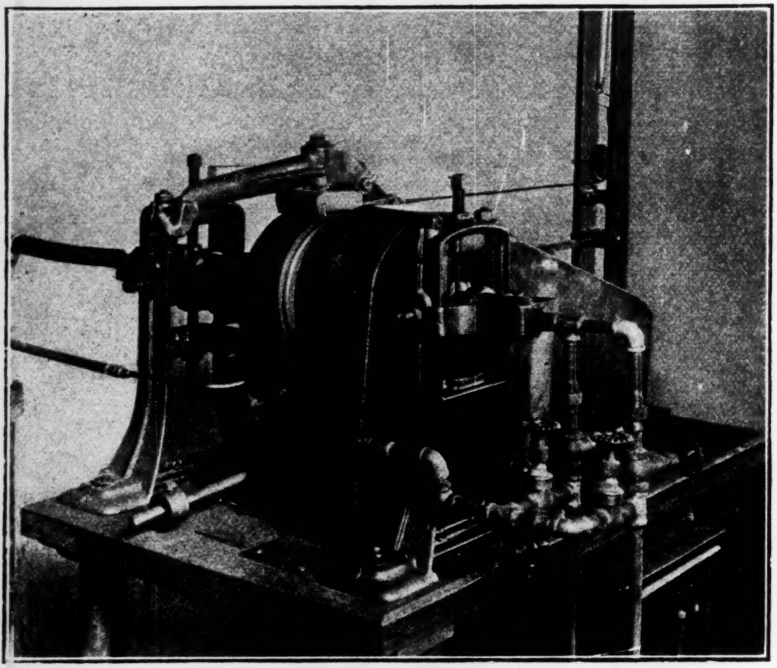

FIG. 1.—Machine Used in Wear Tests.

In all there are many disagreements in results and opinions, caused no doubt, as has been previously mentioned, by the multitude of variables influencing the qualities of cast iron.

As a result of conducting a small investigation into the wear resistance of cast iron, the author has come to realize keenly the difficulty of obtaining useful results from wear testing.

WEAR TESTING OF AUTOMOBILE ENGINE CYLINDERS

Among the complicated ramifications of wear testing are some instances where it is extremely difficult to devise a short-time wear

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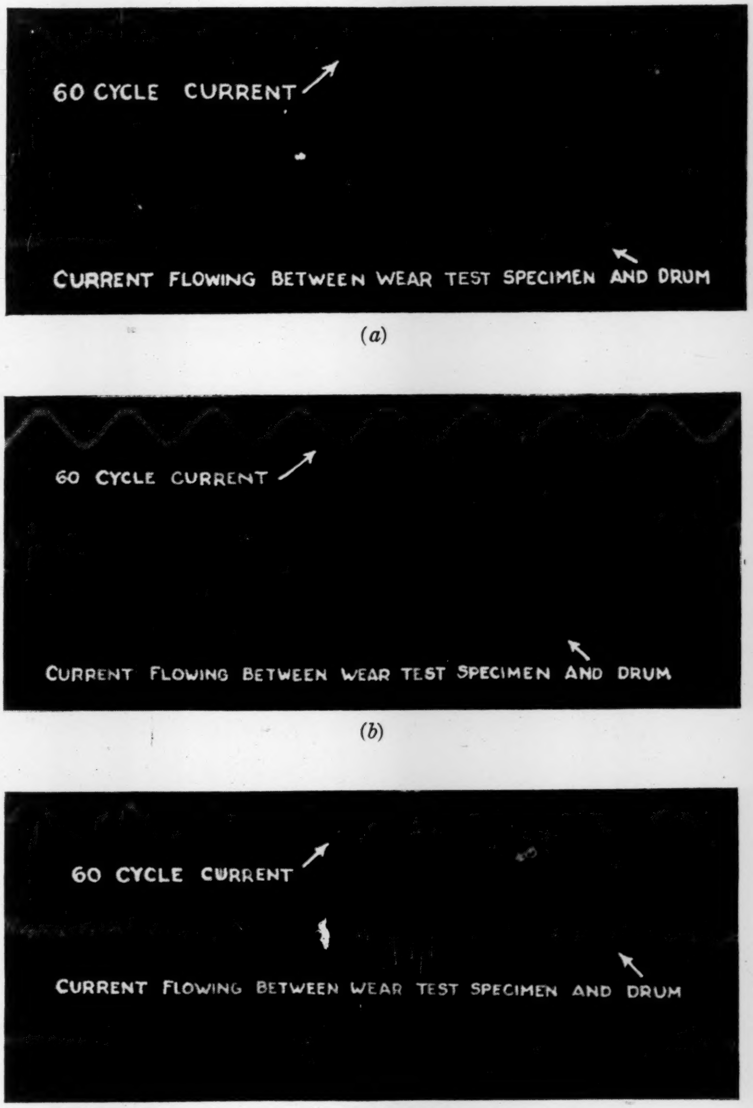
[1] ***Bulletin No. 19***, British Cast Iron Research Assn., January, 1928, Report No. 38 on the Wear of Cast Iron, p. 12.



(*a*)

(*b*)

(*c*)

FIG. 2.—Effect of Pressure on Oil Film.

test which will simulate operating conditions. Take for example the determination of suitability of materials to resist the wear encountered in automobile engine cylinder walls. If the test conditions simulated engine operating conditions faithfully, there should be oil supplied to the surfaces in contact. The question arises of how to control the amount of oil supplied so that one test will be comparable with another. If there is a perfect film of oil between the sliding surfaces, obviously no wear can take place because of no contact of the sliding metals with each other.

Figure 2 (*a*) shows an oscillograph of current in a 5-volt circuit connected to two lubricated sliding metals when a perfect oil film existed between them. The galvanometer showed no deflection, indicating that there was no metal to metal contact to allow passage of the current. This test was made in connection with wear testing of cast iron on a machine depicted in Fig. 1 where a $\frac{3}{16}$ by $2\frac{1}{2}$-in. surface of the test specimen is held in contact with a drum 8 in. in diameter with a $2\frac{1}{2}$-in. face rotating at 800 r.p.m.

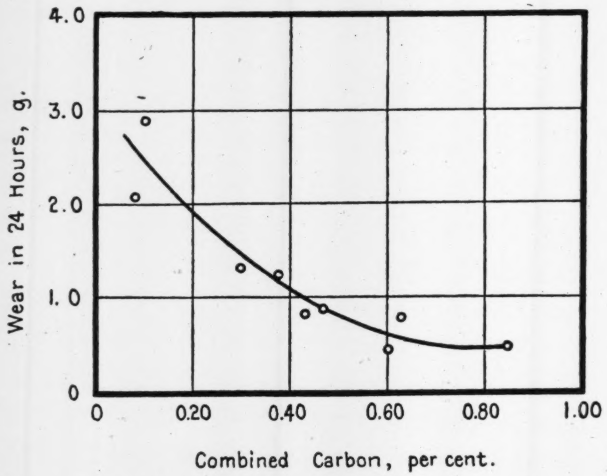
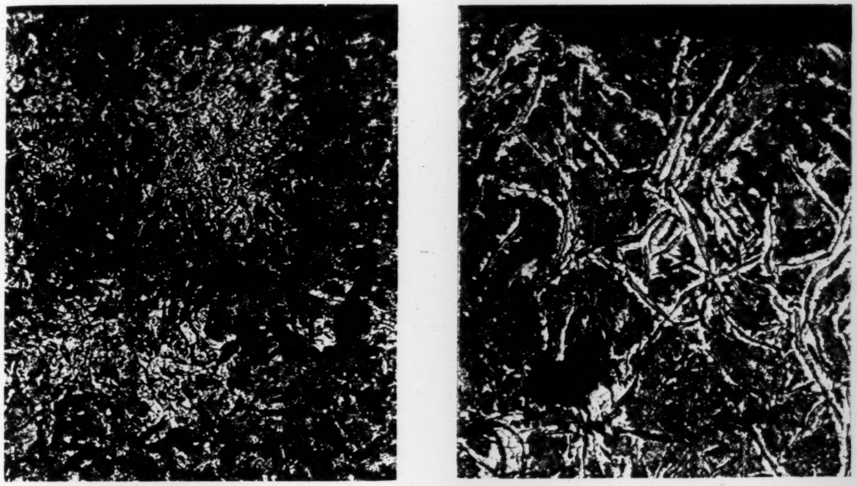


FIG. 3.—Influence of Combined Carbon on Wear of Cast Iron when in Contact with Cast Iron of Brinell Hardness No. 180, and 0.46 per cent Combined Carbon, Sliding at 1600 ft. per Minute Under a Load of 12 lb. per sq. in.

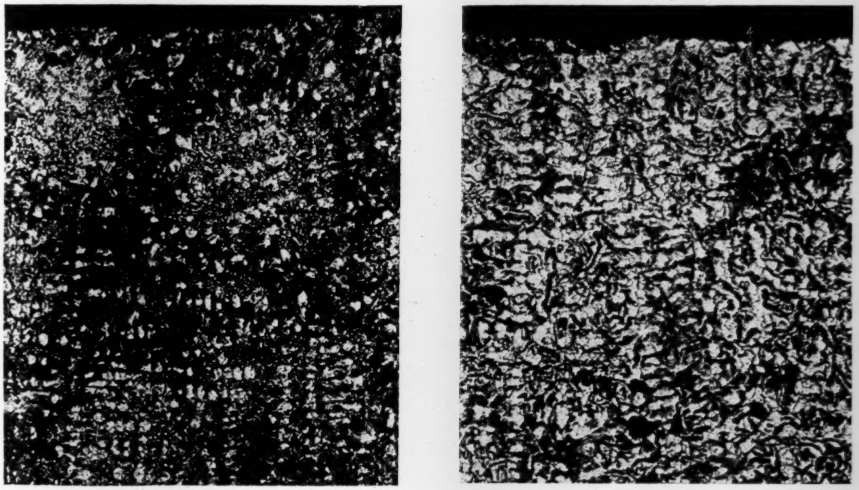
Each point is the average of ten tests.

By increasing the load on the specimens the oil film was partially destroyed, giving a flow of current depicted in the oscillograph as Fig. 2 (*b*). It will be seen here that there is a continual fluctuation of the current, indicating that the oil film is constantly breaking and re-forming. When the oil film breaks there is metal to metal contact



(*a*) Sand cast, Brinell hardness No. 212.

(*b*) Sand cast, Brinell hardness No. 121.



(*c*) Permanent mold cast, unannealed, Brinell hardness No. 223.

(*d*) Permanent mold cast, annealed, Brinell hardness No. 185.

FIG. 4.—Cylinder Irons used in Wear Test (× 100).

at which time wear would occur, but due to the irregular periods of contact no two tests run under such conditions could be compared.

By increasing the load still more, metal to metal contact is continuous as indicated by the oscillograph in Fig. 2 (*c*). Tests run under such conditions would presumably be comparable but it so happens that an attempt to run a test under these conditions produced wear at such a slow rate that it would take a long time to study any number of metal combinations. A substitution of kerosene for the lubricating oil did not speed up the wear sufficiently to render it a practical test. By running the metals together without any lubricant at all, enough wear was produced in a period of 24 hours to indicate the quality of the metal. A series of tests on cast irons of varying combined carbon content gave the information presented in Fig. 3, indicating that greatest resistance to wear is to be found in cast irons containing in the neighborhood of 0.75 per cent combined carbon in sand-cast gray iron. The high combined carbon of this particular iron was due to being melted in an electric furnace. Permanent-mold cast gray iron wore at a rate many times as fast as sand-cast iron of the same combined carbon content. This result is not in agreement with Dudley's[1] generalization that metals with a finer granular structure will wear more slowly. Having obtained this information as to the wear resistance of cast iron unlubricated, the answer was still missing to the question whether wear in the presence of any oil at all takes place in the same manner as that occurring in the absence of oil.

To check up wear resistance of various cast irons in an engine, the following test was made. An air-cooled engine with individual cylinders was used. Each of the four cylinders was made of a different kind of iron, as follows:

(*a*) Sand-Cast Iron, unannealed, 0.51 per cent combined carbon, Brinell hardness No. 212,
(*b*) Soft Sand-Cast Iron, 0.40 per cent combined carbon, Brinell hardness No. 121,
(*c*) Permanent-Mold Cast Iron, unannealed, 0.38 per cent combined carbon, Brinell hardness No. 223, and
(*d*) Permanent-Mold Cast Iron, annealed, 0.06 per cent combined carbon, Brinell hardness No. 185

The microstructure of these four irons is shown in Fig. 4.

The cylinders were assembled in an engine and measured. This engine was placed in a chassis and run 20,000 miles over all kinds of roads, after which the cylinder bores were measured again. There was not over 0.002 in. wear in any cylinder and the minimum wear was 0.0015 in., showing that the lubrication must have been good and that being the case the quality of iron used was not important. Another road test to determine wear of cylinders was made by measuring the cylinder bores of eight cars of different manufacture. The only information about these cylinder bores that was obtained was the chemical analysis. The cars were run 20,000 miles and the bores remeasured. It was interesting to note that the wear decreased with the silicon content, being a maximum of 0.0135 in. in a block containing 2.82 per cent of silicon and only 0.001 in. in a block containing 2.02 per cent of silicon. This result is similar to that obtained by Brinell in studying the effect of silicon.

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[1] C. B. Dudley, "The Wear of Metal as Influenced by Its Chemical and Physical Properties," *Transactions*, Inst. Mining Engrs., Vol. 19 (1890).

Other tests on automobile engines where measured amounts of graded sand were fed into the combustion chamber of running engines showed that wear can be induced in any desired amount by controlling the amount of this abrasive material entering the cylinders.

CONCLUSIONS

The thoughts expressed in this paper can be summarized briefly as follows:

1. A universal wear test can be placed in the same category as the perpetual motion machine.
2. Conditions producing wear can in many instances be imitated in laboratory wear tests which will forecast a metal's behavior in service under conditions like those of the test.
3. Tests to determine wear in the presence of lubricants are difficult to make unless it can be assumed that the elimination of lubricant will give accelerated wear of the same kind as produced with oil present.
4. Wear of automobile engine cylinders can be reduced most by proper lubrication and elimination of abrasive materials. Faulty lubrication will be more disastrous to low combined carbon irons than to those high in combined carbon.
5. There is much conflicting evidence in the literature as to the wearing behavior of cast irons. Future investigators of this subject should have first of all as a foundation, a comprehensive knowledge of the intricate nature of cast iron; otherwise the work is likely to be wasted.

[For Discussion on Wear Testing of Cast Iron, see page 166. —ED.]

MACHINEABILITY OF CAST IRON

BY E. J. LOWRY[1]

SYNOPSIS

This paper discusses the factors which influence machineability. Although machineability of cast iron is a function of abrasion, hardness, and ductility, hardness alone cannot be considered as a true indicator of machineability since it does not measure the abrasive quality of the metal. Any influence which tends to eliminate abrasiveness increases machineability—such as annealing, higher silicon content, higher carbon content or the addition of nickel, titanium, or any other softening alloy.

The machineability of cast iron is a function of the strength and the hardness and the ductility of the material. Hardness cannot be regarded as the true indicator of machineability, due to the fact that it does not measure the abrasive quality of the metal. In this we must differentiate between the hardness of the matrix and the hardness of the hardest material present. The solid solution hardness, such as is found in alloy steels, is not particularly detrimental to machineability, but abrasive hardness, such as imbedded sands, free carbides or phosphides are very detrimental to machineability. Skin hardness, chilled edges and fins on a casting cause the abrasiveness to increase and thus detract from the machineability.

Any influence, therefore, which tends to eliminate abrasiveness increases the machineability. These influences, such as annealing at various temperatures, particularly over 950° F., higher silicon content, higher carbon content, the addition of nickel, titanium or any other softening alloy, react upon the hard substances with the exception of burned-in sands. Nickel particularly reduces the chilling action and the thickness of the hard skin, thus reducing the difficulty of machining.

It has been shown by a paper presented by Boston before the American Society for Steel Treating,[2] that tool hardness, angles of grinding the tool, the speeds and feeds have their effect upon machineability. A paper by Bolton[3] discusses the abrasive characteristics of cast iron, and a further paper by Lowry[1] discusses carbon formations and their relation to machineability.

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[1] Consulting Metallurgist. Detroit, Mich.

[2] O. W. Boston, "Machineability of Metals," *Transactions*, Am. Soc. Steel Treating, Vol. XIII, p. 49 (1927).

[3] E. J. Lowry, "Cast Iron in Its Relation to the Automotive Industry," *Journal*, Soc. Automotive Engrs., February, 1927.

The usual method of measuring machineability is not wholly satisfactory but serves its place in the comparative field. This method is a lathe test under standard conditions, measuring the amount of power consumed by means of a continuous watt meter attached to a direct connected motor, producing the motive power for the lathe. The difficulty in obtaining accuracy is caused by castings containing included sand and slag and small voids, which vitally affect the results. Another factor which should be carefully observed is the cleaning and chipping of the casting before placing it to the test. If the castings are not of average condition in this respect in two different tests the results are not comparative and prove misleading.

In discussing hardness and its effect upon machineability, the average hardness and consequently the strength is greatly increased if the hardness of the hardest particles is reduced or distributed in finely divided states throughout the matrix. For instance, a matrix with a Brinell hardness of 100 containing 10 per cent of constituents like free cementite or free carbides, with a Brinell hardness estimated at 1100, would give an average Brinell hardness of 200, which material would be very difficult to machine, whereas, cast iron of 100 per cent matrix with a Brinell hardness of 200 would be relatively easy to machine. It has been shown that cast iron with a Brinell hardness as high as 340 can be machined without difficulty when this hardness is obtained by the use of sorbitizing alloys, such as molybdenum or nickel-chromium combinations.

Annealing is very helpful in increasing machineability, but care must be used in obtaining the correct cycle, because when a casting is annealed at over 1000° F. for any length of time the strength is materially reduced. Below this point, it has very little effect on the strength, but does reduce the resistance to machining.

It should be pointed out that the use of rigid machines and well-chosen tools will overcome some of the machining difficulties which are sometimes attributed to the hardness of the casting. A change in the set-up for the grinding of a tool sometimes overcomes difficulties. Investigation of machineability and the effects of various elements on this property should receive wider attention and should form part of the investigation of cast iron now being carried on by research groups.

[For Discussion on Machineability of Cast Iron, see page 166. —ED.]

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[1] J. W. Bolton, "Factors Influencing the Machineability of Cast Iron," *Machinery*, March, 1925.

CORROSION OF CAST IRON

By H. O. Forrest[1]

Synopsis

The factors affecting the corrosion of cast iron include not only the characteristics of the metal itself but also, to a greater extent, the composition of the corroding medium and the type of protective coating employed. The coatings may be either applied or precipitated upon the metal. Preliminary tests at the Research Laboratory of Applied Chemistry of the Massachusetts Institute of Technology conducted under conditions comparable to service conditions indicate only small differences in the rates of corrosion of cast-iron pipe manufactured by different processes. These indications point to the need for use of proper protective coatings applied before the pipe is put into service or else for treatment of the corroding medium with chemicals to bring about precipitation of protective scales or films. The primary requisites for good coatings are impermeability to water and freedom from pinholes so that the corroding medium cannot reach the metal surface, since a few holes in a pipe are almost as serious as complete corrosion.

Corrosion tests in soils point to the fact that in general the nature of the soil rather than the types of metal of certain classes is the factor which governs the corrosion rate. Cast iron, in the majority of the cases reported, is slightly superior to steel or wrought iron in corrosion resistance in the atmosphere. There are many cases in which cast iron is superior to other ferrous materials, for example, in handling various chemical materials such as strong sulfuric acid, dry sulfur dioxide, organic distillates, etc.

Introduction

In discussing any corrosion problem it is necessary to keep in mind the fact that the chemical composition, physical characteristics and method of manufacture of the metal itself are not the only important factors. The corroding medium and the form of protective coating employed may be of even greater importance and entirely overshadow any variations in the metal. This is particularly true of corrosion under natural water and especially in the case of cast iron where a heavy coat is usually applied. For this reason it is necessary to subdivide the discussion into three phases: the corroding medium, the protective coat, and the metal itself.

Although there are many references in the literature to tests on corrosion of cast iron, the majority do not place sufficient emphasis upon the factors other than the metal itself, and for that reason the results obtained are inconclusive. A tremendous number of isolated cases of extreme corrosion or of exceptionally long life may be recalled by almost anyone who has specialized in water-works practice or in the manufacture or distribution of cast-iron pipe. It is not the purpose here to point to such examples, but merely to discuss what is known of the general laws governing the corrosion of cast iron.

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[1] Director of Research Laboratory of Applied Chemistry, Massachusetts Institute of Technology, Cambridge, Mass.

To limit the discussion still further the major effort will be devoted to those cases where the corroding liquid is natural water. A great deal of controversy might be aroused by considering soil, atmospheric, gas main, chemical and electrolytic corrosion; but since the vast majority of cast-iron pipe is used for water mains, it seems advisable to deal to the greatest extent with that phase.

CORROSION OF CAST IRON IN WATER

The generally accepted electrochemical theory of corrosion states that metal tends to go into solution with the liberation of hydrogen. In order for the reaction to proceed this hydrogen must be removed from the sphere of reaction either by evolution as gas or by oxidation to form water. Except in the case of chemical corrosion, the latter mechanism is the important one and determines the extent to which corrosion will proceed. Although many other factors influence the rate of attack, it would only confuse the main issues to discuss them at this time. It is sufficient to say that in the usual ranges encountered in natural waters, such factors as temperature and velocity are insignificant except as they change the resistance to penetration of oxygen.

In view of these basic facts it is obvious that the means for preventing or decreasing corrosion under water lie in preventing oxygen reaching the surface of the metal, either by removing it from the water or by placing a resistance in its path. The former requires no discussion since, for various reasons, water without oxygen is not desirable and furthermore its removal would entail a tremendous expense. The latter offers the best and perhaps the only economical method of submerged corrosion prevention. A resistance may be inserted in the path of the diffusing oxygen by (1) preparation of the pipe, using a protective coating such as cement or asphalt, or (2) by formation of protective films during use. These latter films may be natural, due to the normal constituents of the water carried and the metal used, or artificially formed by treatment of the water.

Protective Coatings:

Unfortunately very little information is available on the protective effect of coating materials. An essentially non-conducting layer, such as cement or bituminous material, would be entirely successful if absolutely impervious. The major difficulty with such coatings lies in pinholes or breakage, due to brittleness, permitting corrosion to take place and forcing the layers away from the wall. Under such conditions protection is lost, since corrosion will proceed; and a few holes in a pipe are almost as serious as complete corrosion of the entire surface. From the view point of maintenance of carrying capacity, however, the protective coating is of undoubted value.

So far as differentiation between different types of coating is concerned, the important factor is flexibility and resistance to oxidation. A material which is subject to cracking or chipping during transportation previous to use is not as satisfactory as one which is flexible. An asphalt which oxidizes rapidly is not as satisfactory as one which resists oxidation. This later factor is not so vitally important, however, since Speller[1] reports an oxidation to a depth of only $\frac{1}{64}$ in. after exposure to acid soil for five years. The use of such coatings as bituminous material and cement is of great importance and should result in considerably increasing the life of pipe when properly applied to a thoroughly cleaned surface and maintained free from pinholes and cracks. The many divergent opinions on the value of this method of corrosion prevention are undoubtedly due to differences in the continuity of the coating.

In addition to coatings which are applied before use, it is possible to insert a resistance to the path of oxygen while the corrosion process is taking place. This is well illustrated by the work of Baylis[2] in treatment of the water supply of the City of Baltimore, when the artificially hardened water produced much lower corrosion. It is also illustrated by the experience of a city in England, where the entire system is treated with silicate. Still another example is indicated by the difference in condition in those sections of New York City served by the Croton supply as contrasted with those served by the Catskill supply.

Results of a great number of investigators during the past few years have indicated that the formation of a protective film by adding such materials as lime or silicate to the water have been effective in reducing the corrosion rate to a small fraction of its previous value. For example, Thresh[3] found sodium-silicate additions to water to decrease the corrosion of lead. Whitman and co-workers[4] report effective prevention of condenser corrosion in fresh-water re-circulating systems by the addition of 20 p.p.m. of 40° Baumé sodium silicate to the water. Russell[1] used sodium silicate for prevention of red water in hot-water heaters. Speller[2] successfully reduced the corrosion rate in hot Pittsburgh water by the addition of 20 to 50 p.p.m. sodium silicate. Roberts[3] reports the use of sodium silicate as a corrosion retarder in fresh-water condenser systems. Baylis[4] successfully treated Baltimore city water with lime in quantities sufficient to obtain a protective carbonate film on the inside of water mains. Forrest[5] and co-workers showed that lime additions to brackish waters having an appreciable bicarbonate hardness caused the formation of protective carbonate films on iron and steel.

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[1] F. N. Speller, "Corrosion, Causes and Prevention ' McGraw-Hill Book Co. (1925), p. 534.

[2] J. R. Baylis, *Journal*, Am. Water Works Assn., Vol. 15, p. 598 (1926).

[3] J. C. Thresh, *Analyst*, Vol. 47, p. 459 (1922).

[4] W. G. Whitman, C. L. Chappell and J. K. Roberts, *Refrigerating Engineering*, Vol. 12, p. 158 (1925)

The purpose of the above discussion is to bring forcibly to mind the great effect on the corrosion of metal of essentially non-porous non-conducting films. By their use the rate of corrosion of metal in water may be enormously reduced and they are therefore to be recommended in a wide majority of cases. One very important factor must always be kept in mind, and that is absolute continuity of the protective coating.

It may seem that undue emphasis has been placed on the effects of factors other than the type of cast iron itself in submerged corrosion. Such emphasis is, however, fully merited, since these factors are capable of causing vastly greater differences in corrosion rate than any variations in the metal itself, within the range of usual practice. As an exaggerated example we may conceive of a cast-iron pipe perfectly coated with asphalt as compared with a bare pipe in corrosive water. It is almost impossible to imagine a type of bare cast iron which would have a zero corrosion rate as would the pipe with the perfect asphalt coating.

Influence of Characteristics of Cast Iron on Corrosion:

Although the resistance offered to oxygen diffusion is a major factor, it seems advisable to briefly refer to the available information on the effect of variations in the metal itself. During the last century a great many tests have been conducted to determine the corrosion-resistant properties of iron and steel. There are essentially no data on the comparative life of different types of cast iron.

The factors concerned with the metal itself which influence the corrosion of cast iron may be divided into two general classes: surface characteristics and internal characteristics. The former are influenced by method of casting, including the type of process, kind of facing, chill, etc., while the latter are influenced chiefly by the ore used and the treatment prior to casting and include chemical composition, grain size, dispersion of metalloids, etc.

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[1] R. P. Russell, *American Dyestuff Reporter*, Vol. 15, p. 61 (1926).

[2] F. N. Speller, *loc. cit.*, p. 349.

[3] J. K. Roberts, H. O. Forrest and R. P. Russell, *Refrigerating Engineering*, Vol. 14, p. 173 (1927).

[4] J. R. Baylis, *Journal*, Am. Water Works Assn., Vol. 15, p. 598 (1926).

[5] H. O. Forrest, J. K. Roberts and B. E. Roetheli, *Journal of Industrial and Engineering Chemistry*, Vol. 21, p. 33 (1929).

There are numerous references regarding the effect of surface differences of ferrous metals on corrosion rates, electrode potentials, overvoltage, and pitting. In aqueous corrosion, mill scale on steel[1] tends to act as a cathodic or depolarizing area which localizes the attack on the uncovered portions of the metal. Exposed graphitic particles tend to accelerate the corrosion of cast iron in acids while rough surfaces tend to lower the overvoltages of ferrous metals and bring about increased solution rates.[2]

TABLE I.—REPORTED DATA ON PROTECTIVE EFFECT OF FOUNDRY SKIN ON CAST IRON.

Material	Clear Sea Water	Foul Sea Water	Clear River Water	Foul River Water
RATES OF CORROSION OF CAST IRON, CM. PER YEAR R. Mallet, *Journal*, New England Water Works Assn., Vol. 42, No. 3, p. 261 (1840).				
Cast iron with skin removed	0.0117	0.0031	0.0014	0.0061
Cast iron with skin	0.0096	0.0192	0.0036	0.0097
Chilled cast iron	0.0089	0.0087	0.0018	0.0070
RELATIVE CORRODIBILITIES OF CAST IRON IN NEUTRAL MEDIA B. H. Thwaite, *Journal*, Iron and Steel Inst., No. II, p. 667 (1880).				
Cast iron with skin removed	191	100	261	100
Cast iron with skin	270	350	256	188
Galvanized cast iron	115	120	47	97

Various data have been reported on the protective effects of foundry skin on cast iron. Some of these are given in Table I. These results are typical of the general status of the literature on cast iron corrosion, in that they do not agree. The major reason for this is that most of the available data on corrosion of cast iron have been obtained either by accelerated tests or by long-time exposure under conditions which were not adequately controlled. When a sub-committee of the Sectional Committee on Specifications for Cast-Iron Pipe was faced with the problem of determining the effect of these factors it was necessary to institute a program of research. The experimental work is being carried out at the Research Laboratory of Applied Chemistry at the Massachusetts Institute of Technology, and although the results of all tests are not entirely complete, it seems advisable at this time to discuss part of them from a general standpoint.

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[1] F. N. Speller, *loc. cit.*, p. 31.

[2] M. Knobel, *Transactions*, Am. Electrochemical Soc., Vol. 47, p. 131 (1925).

A laboratory method of testing has been adopted for this work, with an attempt to simulate, as nearly as possible, actual working conditions. Small samples cut from standard 8-in. pipe are rotated in constantly renewed Cambridge water at a speed comparable with the flow of water through pipes. A number of samples of each kind is used and the loss in weight and extent of pitting determined at intervals up to 300 days. New samples are used for each point on the curve showing the relation of loss of weight to time, and the slope of the line after an initial period of 80 days is taken as the corrosion rate.

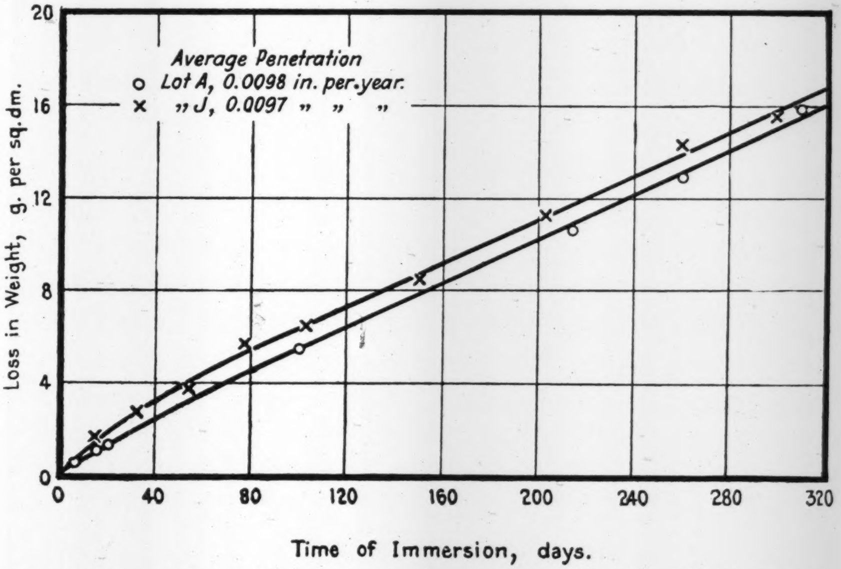
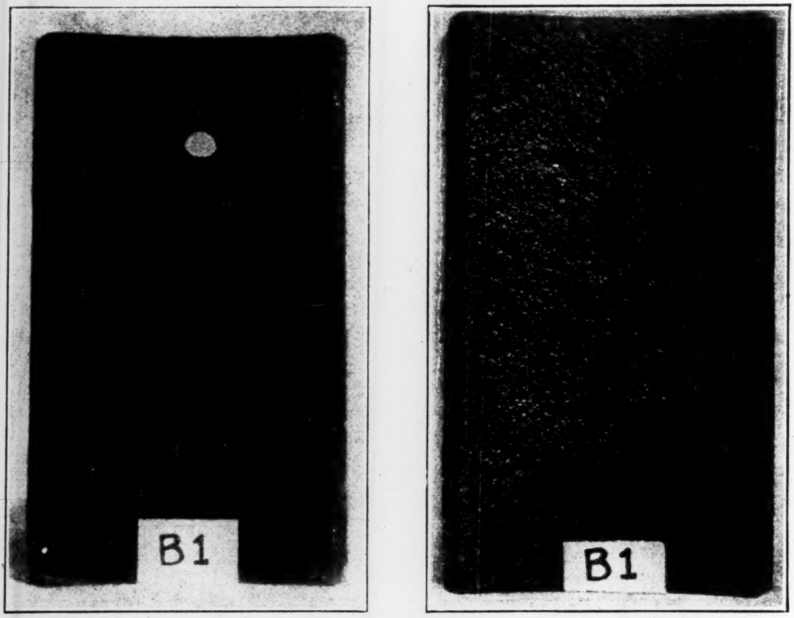
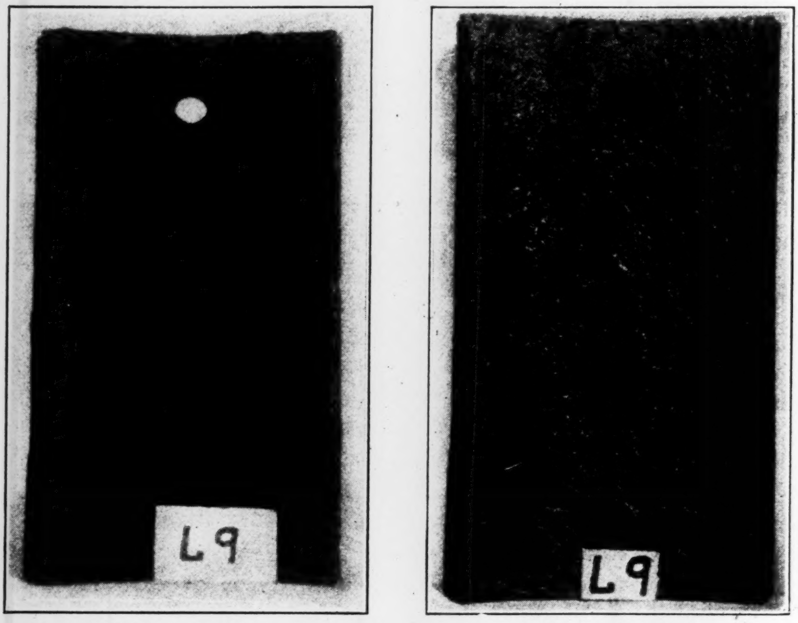


FIG. 1.—Corrosion of Cast Iron in Cambridge Water at 68° F. Water Velocity Through Jars, 4.55 gal. per hour.

The chief advantage of this method of testing is that it allows absolute control of all variables and takes into account the type of rust film formed. Temperature, velocity, oxygen concentration and chemical composition of the water are all maintained constant, so that the only variable in the tests is the metal itself. Up to the present time, including tests to 300 days on a number of samples, the slope of the line has remained constant after an initial period of 80 days. It is conceivable, however, that the slope on some samples may decrease at a still later date.



(*a*) Before Cleaning. (*b*) After Cleaning.



(*c*) Before Cleaning. (*d*) After Cleaning.

FIG. 2.—Specimens Before and After Cleaning, Showing the Extent of Attack.

Corrosion rates have now been determined for a number of samples of cast iron made by different processes and covering the normal ranges of chemical composition for time intervals up to 300 days. Fig. 1 shows the type of results obtained on two samples of cast iron and indicates the necessity for continuing the tests longer than the initial period of 80 days. The photographs reproduced in Fig. 2 show types of rust films formed on two types of cast iron. Such photographs are being taken of all the long-time exposure samples, before and after cleaning, to give a visual picture of the extent of pitting.

Although it is still somewhat early to state the effect of all the different variables in the composition and manufacture of cast iron on the corrosion rate, there are a few general observations which should be mentioned in connection with the present discussion. It must be remembered that only twelve samples have been tested to the 300-day period and that there may be minor changes in the conclusions when all tests are complete.

The average corrosion rate of the twelve normal cast-iron samples is approximately 0.01 in. per year with a minimum rate of 0.0090 and a maximum rate of 0.0117 in. per year. The average deviation from the mean value for all samples was only 7 per cent. Assuming uniform corrosion, this means that the life of pipe would be expected to vary only about 7 per cent with variations in composition and method of manufacture. In all cases, however, some pitting takes place; but in general the depth of maximum pit is not over twice the average corrosion, while in a few exceptional cases it is as high as four times. These cases have not been fully investigated up to the present time but will be before the tests are completed. Preliminary results indicate that high silicon (over 4 per cent) and high nickel (over 4 per cent) considerably decrease corrosion rate, but that within the usual range the effect is not great.

From the results of this investigation it is evident that the rates of corrosion of the usual types of cast iron made in this country do not vary sufficiently to cause any great differences in length of life. It may be possible by the introduction of alloy cast irons such as high silicon (4 per cent) or high nickel (up to 14 per cent) to markedly change the corrosion resistance. So far as the normal irons are concerned, however, the difference is so small as to be negligible. It may be concluded, therefore, that for submerged natural water, corrosion is mostly affected by the type and quality of the protective coating and by the protective films formed during use.

OTHER TYPES OF CORROSION

Soil Corrosion:

In addition to aqueous corrosion problems several other types arise due to the varied number of conditions to which cast iron is exposed. One of the most important of these is soil corrosion. Any corrosion taking place on the outside of buried pipes, such as are found in water, gas and oil lines buried in the soil, comes under the general heading of soil corrosion. This includes both natural and electrolytic corrosion. The subject was given but little study prior to 1922 when the U. S. Bureau of Standards inaugurated extensive tests on natural soil corrosion. Tests to date in 47 different soils have shown that different kinds of metal of the same type behave similarly in the same type of soil, whereas wide variations exist between the corrosion rate of certain groups of metals in different soils. Since the inauguration of the U. S. Bureau of Standards tests, the American Petroleum Institute and the American Gas Association have commenced work on researches for the purpose of finding good protective coatings for underground pipe. The fine cooperation between these large industries promises to result in a speedy solution of the underground corrosion problem.

TABLE II.—ATMOSPHERIC CORROSION OF CAST IRON.

K. Arndt, *Chemiker Zeitung*, Vol. 34, pp. 425 and 1078 (1910).
Values reported are number of cubic centimeters of O_2 absorbed.

Days	Cast Iron	Ingot Iron	Steel
3	39	42	80
10	83	168	269
30	161	437	549
50	221	596	721
122	416	949	1132

TABLE III.—CORROSION IN THE ATMOSPHERE.

James Aston and G. K. Burgess, Eighth Internat. Congress of Applied Chemistry, New York City, Vol. 26, p. 453 (1912).

Material	Average Penetration, mils per year	Pitting Factor	Product of Columns 2 and 3
Bessemer Steel	1.83	5.45	9.97
Swedish Wrought Iron	2.08	4.57	9.50
Open-hearth Pure Iron	2.29	4.76	10.9
Cast Iron	2.64	3.51	9.25
Open-hearth Steel	2.84	3.76	10.70

The problem of stray current electrolysis has in a large measure been solved by the now almost universal use of alternating current.

The chief sources of stray currents to-day are the direct-current street railway systems. However, by adequate insulation of return leads the occurrence of stray current corrosion has become the exception rather than the rule.

Atmospheric Corrosion:

Corrosion of cast iron in the atmosphere has been investigated in several instances. The majority of tests to date show cast iron to be slightly more resistant in the atmosphere than steel or wrought iron. Some typical cases are given in Tables II and III.

Oil and Gas Corrosion:

Another important problem arising in the corrosion of irons is oil and gas main corrosion. It has been pointed out that in the majority of cases in which gas main corrosion is serious it is well to look for hydrogen sulfide or sulfur dioxide as the chief source.[1] These constituents in the presence of traces of moisture exert a great accelerating effect on corrosion. Hydrogen sulfide forms a sulfide of iron, and if sulfur dioxide is present as well, the sulfurous acid formed continually dissolves off the sulfide film thus accelerating corrosion. It is therefore obviously desirable to keep sulfur in any form at as low a value as possible in the gas. Oil mains suffer a similar action due to the large quantities of sulfur present in some of the crude oils.[2]

In view of its physical properties, cast iron is frequently used in chemical plants; however, it is in general less resistant to dilute acids than steel or wrought iron.[3] In caustic solutions cast iron stands up well in low alkali concentrations, but goes into solution much more rapidly than iron or steel in fused alkalies. Cast iron has been used successfully in many phases of chemical industry. It is shown by Speller[4] that cast iron is satisfactory for organic acid stills, nitric acid stills, drums for strong sulfuric acid (85 to 100 per cent), piping sulfur dioxide gas at temperatures higher than 200° C., bleach liquor piping, piping dry chlorine gas, chlorine solutions, black liquor evaporators and for many other uses.

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[1] W. V. Watson, *Gas Age*, Vol. 54, p. 557 (1924).

[2] R. Van A. Mills, U. S. Bureau of Mines *Bulletin 233* (1925).

[3] E. Heyn and O. Bauer, *Mitteilungen* aus dem Königlichen Materialprüfungsamt, Vol. 28 (1910).

[4] F. N. Speller, *loc. cit.*, pp. 582–588.

[For Discussion on Corrosion of Cast Iron, see page 169.—Ed.]

HEAT TREATMENT OF CAST IRON

By F. B. Coyle[1]

Synopsis

The paper discusses some of the important features involved in the heat treatment of cast iron. The discussion for convenience has been divided into three temperature ranges: (1) up to 800° F. within which there is no change in structure or physical properties; (2) 800° F. to the critical range within which there is a change in structure but no change in physical properties; (3) from the critical range to the melting point within which both structure and physical properties change.

The author refers to an extensive digest of the literature of the heat treatment of cast iron which is being prepared by the Society's Committee A-3 on Cast Iron and expresses his belief that systematic investigation of this field will disclose that the applicability of heat treatment to cast iron will be of distinct economic value among the industries.

The thought has often been expressed that if alloying elements could produce considerable increase in relative value of properties of steel, alloying elements should produce a like improvement in cast iron. Since cast iron is essentially a steel whose continuity is broken up by free graphite, this deduction is not unreasonable and indeed seems justified by a recent survey of the alloy cast-iron industry.[2] Since the intrinsic value of alloy steel is only realized through heat treatment, it is reasonable to assume that cast iron in general should be amenable to heat treatment processes.

Even a brief survey of some of the literature of cast iron demonstrates the fact that not only has considerable time and effort been devoted to the study of the heat treatment of cast iron, but that the subject is an extremely broad and complicated one for study. A survey conducted by Committee A-3 on Cast Iron of this Society[3] has shown that there is sufficient interest among the various industries to warrant a systematic study of the possibilities of heat treatment of cast iron.

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[1] Research Metallurgist, International Nickel Co., Bayonne, N. J.

[2] E. F. Cone, "Ascendency of Alloy Iron Castings," *The Iron Age*, March 28, 1929.

[3] Report of Sub-Committee XVI, of Committee A-3, on Heat Treatment of Cast Iron, see *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part I, p. 125 (1929).

There are four important factors to be considered in subjecting cast iron to heat, which are fully recognized among the industries. These are:

1. *Artificial Aging.*—Internal stresses set up in castings during cooling because of variations in contour and thickness. Heating to a relatively elevated temperature relieves these stresses and prevents distortion of the casting after machining or during subsequent service.

2. *Alteration of Chemical and Physical Properties.*—By annealing at temperatures above the critical range it is possible to reduce hardness considerably and thereby increase machineability. This practice has been resorted to on a large scale.

3. *Growth.*—It is well known that most grades of cast iron increase in size permanently when subjected to elevated temperatures. This is a serious problem in many instances, and the elimination of growth will greatly increase the applicability of cast iron.

4. *Rust Proofing.*—The processes of parkerizing, galvanizing and tinning have been applied to gray iron castings for this purpose. All of these processes involve elevated temperatures and some give rise to considerable difficulties.

While these factors illustrate the importance of the subject under consideration, the following brief analysis will give a general conception of the broad scope and extended possibilities of this field of study.

The subject can conveniently be divided into three fields according to temperature range, as follows:

1. Up to 800° F.—no change in structure or physical properties occurs.

2. From 800° F. to the critical range—a change in a structure occurs, but no change in physical properties.

3. From the critical range to the melting point—both structure and physical properties change.

Temperature Range up to 800° F.:

Growth.—Permanent increase in size is effected when cast iron is subjected to certain types of carbonaceous gases. Such gases penetrate the metal and through a process of cracking deposit free carbon. Graphitization followed by growth is also caused by corrosion due to exposure to such media as sea water.

Aging.—A slight relief of casting stresses is effected when castings are subjected to temperatures slightly under 800° F. for long periods of time.

Physical Properties.—Some investigators have found that when cast iron is exposed to some gases at temperatures slightly below

800° F. the strength is materially diminished. In general, however, cast iron maintains its strength uniformly up to 800° F.

Temperature Range from 800° F. to the Critical Range:

Structure.—Several investigators have shown definitely that cementite of pearlite decomposes with the accompanying precipitation of graphite within this temperature range. It has also been shown that complete malleablization has been produced within this temperature range on a laboratory scale. It is known that the alloying elements nickel and chromium inhibit this change in structure, but the published data are very meager. Further work is required to demonstrate the exact effect of these elements and the effect of other alloying elements on this particular change.

Aging.—Artificial aging can be completely effected within this temperature range. However, since the graphitization which also occurs reduces both hardness and strength, it is advisable to restrict the aging treatment to temperatures below 950° F.

Growth.—Within this temperature range growth is initiated by graphitization which also produces incipient cracking. In the presence of oxidizing gases growth is aggravated by the gases penetrating along graphite flakes and oxidizing silicon to silica. Carbonaceous gases also enter the metal through the graphite flakes and by cracking deposit more graphite and further fissuring results until the metal is finally affected throughout.

Temperature Range from the Critical Range to the Melting Point:

Physical Properties.—The change in physical structure of the metallic ground mass which occurs is similar to that which occurs in steel.

Structure.—Within the lower portion of this temperature range the change in structure of gray iron is identical with that which occurs below the critical range. However, as the temperature increases this change is diminished due to the power of the metal to redissolve carbon as the temperature increases. Of course the exact mechanism of the change in structure depends upon the initial structure of the metal. An iron which is initially white can be completely graphitized (malleablized) if free of alloying elements, or only partially graphitized if alloying elements are present. The range of physical properties obtainable in this manner is considerable.

Annealing.—Annealing at temperatures slightly above the critical range reduces hardness and increases machineability by decreasing combined carbon. However, strength is also diminished and iron subjected to such treatment is extremely weak.

Growth.—Growth occurs within this temperature range in the same manner as described for the preceding range, but is aggravated and more rapidly accomplished. The use of alloying elements shows promise of reducing or eliminating such growth, but considerable investigation yet remains to be done.

Carburizing.—Carburizing followed by subsequent hardening has actually been resorted to for developing hard wearing surfaces for special applications.

Quenching.—Quenching has been carried out in several instances in order to accomplish the same purpose as described under carburizing.

Quenching and Drawing.—This treatment, in so far as published data indicate, shows little promise when applied to plain cast iron, but its usefulness may be possible with alloy cast iron. This treatment has been demonstrated to effect an increase in strength of 50 per cent or more when applied to low-carbon iron or high-strength cast iron.

CONCLUSION

In this brief presentation, the author has only been able to review the subject of heat treatment of cast iron in an extremely general manner. Much has been written describing particular applications, but nothing had been done to correlate such data with respect to the whole subject until Committee A-3 of the Society began the preparation of a digest of literature. It is expected that this digest will be ready for publication by the Society in 1930.

It only remains for the author to express his profound belief that systematic coordinated investigation in the field of study under consideration will disclose that the applicability of heat treatment to cast iron will be of distinct economic value among the industries.

DISCUSSION ON CAST IRON

Classification and Properties of Cast Iron

Mr. J. S. Shaw[1] (*by letter*).—Bolton's classification is useful and his volume-surface area ingenious. How far the latter can be applied to large complicated castings is doubtful. The large task he has set himself (some 200 different mixtures), if completed, can but add to our knowledge of cast iron. In judging his results, however, it must be kept in mind that apart from analysis and cooling rates, he has asked that other factors should be considered as established. How important these factors are is being investigated here in England where the problem is being studied from another angle: namely, why do irons of the same analysis and under similar cooling conditions yield such variable test results? Progress is being made and the following analyses and test results show the effect of steel additions, altered cupola conditions and casting temperatures on irons of practically the same analyses and cooling conditions. The test bars were all 14 by 1 by 1 in. The following table gives three low, three medium and three high tensile strength results from a series of 150 bars taken from work extending over 12 months. The cupola conditions were such that might prevail in various foundries and the resultant metal was all run into salable castings:

Tensile Strength, lb. per sq. in.	Shear on 0.5-in. diameter, lb. per sq. in.	Total Carbon, per cent	Silicon, per cent	Total Carbon +Silicon, per cent	Total Carbon +0.30 per cent Silicon, per cent	Ratio, Shear to Tensile Strength	Combined Carbon, per cent	Manganese, per cent	Sulfur, per cent	Phosphorus, per cent
22 000	37 600	3.44	1.56	5.00	3.91	1.71	0.59	0.68	0.11	0.64
22 400	36 300	3.46	1.60	5.06	3.94	1.62	0.56	0.65	0.114	0.48
24 400	40 800	3.41	1.93	5.34	3.99	1.67	0.51	0.76	0.103	0.55
30 000	44 800	3.43	1.49	4.92	3.88	1.49	0.63	0.71	0.113	0.64
30 200	44 600	3.44	2.03	5.47	4.05	1.47	0.49	0.72	0.089	0.86
33 400	45 700	3.43	1.98	5.41	4.02	1.37	0.53	0.74	0.11	0.61
39 400	54 900	3.35	1.49	4.84	3.80	1.39	0.60	0.76	0.12	0.51
40 500	60 500	3.36	1.47	4.83	3.80	1.49	0.56	1.20	0.103	0.58
42 100	41 700	3.43	1.91	5.06	4.00	1.00	0.58	0.72	0.116	0.76

These results can be reproduced within practical limits, and the micrographs show that the test results are dependent on graphite size and formation together with the formation of the matrix.

It is somewhat difficult to gather what object Mr. MacPherran had in mind when he submits three analyses and cast bars varying in diameter from 1¼ to 4 in. out of the same ladle. It is evident at once that any one analysis is not suitable to the various cooling rates of these bars or that reasonable tests can be expected in all cases. From personal knowledge I am sure Mr. MacPherran would be the last man to pour castings averaging 1 in. thick and others averaging 4 in. thick from the same ladle, especially if they had to stand a water test. Why then do this with test bars and further why turn 4 in. down to 0.5 in. and expect a result that is in keeping with a test when the 4-in. bar is tested at 3.75-in. diameter.

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[1] Southsea, England.

This is the fundamental difference between British and American investigation. Here we say make your test bar diameter bear a near relation to the average section of the casting it represents. Cast it with a suitable metal for that thickness. Take as little off the diameter of the bar in the center that will ensure breakage away from the grips. Under these conditions, with other factors stabilized, valuable information is obtained. Mr. MacPherran's results only tend to "Soil our own nest."

With regard to the high-test iron, are we to attribute the uniform hardness of this metal to the low carbon content of 2.55 per cent or to the 1.43 per cent nickel? At the recent London Exhibition three 6-in. diameter blocks were shown on different stands. In each case there was very little difference in the Brinell hardness right across the face. In one case this was attributed to the use of nickel, in a second to low silicon and hot mold, while the third attributed it to low carbon and steel additions. It is claimed that nickel additions while raising the Brinell hardness considerably do not affect the machining speeds, due it is stated to a finer graphite and matrix. So it would seem Brinell hardness numbers are not a true index of hardness.

Messrs. Rother and Mazurie refer to their valuable piece of work in 1926 on transverse bars. But here again if the 3-in. bar had been at its strongest there is little doubt the 1-in. bar would have contained free cementite and the transverse results would probably have been much lower and the percentage difference between the 1-in. and 3-in. bars be much lower, if not even reversed. It has been clearly proved that no fixed ratio exists between either tensile strength, transverse strength, or shear. It is true that with careful control of raw material, cupola practice, casting and cooling conditions, some rough ratio may be found. Both the I. B. F. and the British Engineering Standards Association committees endorse Mr. Rother's view regarding the use of the modulus of rupture figure, and consider its use a dangerous one in view of the well-known difference in the breaking loads per square inch of thin and thick bars of similar composition.

Regarding Mr. Coyle's contribution, I have little sympathy with the use of Maurer's diagram or any other combination of total carbon plus silicon. These may be of service to university students or young metallurgists but of little practical utility to a man who knows his job. It is now generally taken for granted that for high-strength iron a low carbon is needed. This is fixed by the cupola practice at anything from 2.8 per cent to 3.2 per cent. The calculated silicon content then depends on the thickness and design of the casting. If the cupola cannot be controlled, the diagram is of no use. If it can be, a diagram is not needed. If Mr. Coyle will abstract the whole of Maurer's figures and attach the tension test results which are plotted on a later diagram in the same paper, he will find with the same total carbon plus silicon and each with an all pearlitic structure, tension tests on the same size bar varying from 21,800 to 42,300 lb. per sq. in. Or glance at the table mentioned earlier showing what a marked effect other factors than total carbon plus silicon have on the product.

MR. F. B. COYLE[1] (*author's closure by letter*).—It is difficult to conceive Mr. Shaw's connection between the Maurer diagram with a systematic relation of total carbon plus silicon. No mention of such a connection was made in the paper. The diagrams are chiefly of educational value in that they express fundamental facts. In this they accomplished the purpose of the symposium, which was to place fundamental data in the hands of engineers and designers. These data, as presented, refer to properties obtained in arbitration bars cast under standard conditions. They were intended to show relative effects of variations in compositions. They also show that no simple systematic relation between strength and composition exists.

Mr. Shaw advises that the strength figures of Fig. 2 and subsequent diagrams should be referred to or super-imposed upon the Maurer diagram. If Mr. Shaw had inspected the paper closely, he would have seen that this had been done, since in each figure the original Maurer diagram is represented by broken lines. Comparison of the columns showing total carbon plus silicon and tensile strength in Mr. Shaw's table confirms the presentation in the diagrams that there is no simple relation between composition and strength. There is no mention in the paper that the diagrams, as presented, are practical guides for the manufacture of castings. The data must be modified to meet manufacturing conditions; that is, known knowledge of correlation of test bar with casting must be used in conjunction with the diagrams, in order to render them applicable to manufacturing conditions. It is needless to say that if any foundryman does not use the best cupola practice, all the data in the world will be of no value to him.

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[1] Research Metallurgist, International Nickel Co., Bayonne, N. J.

MR. R. S. MACPHERRAN[1] (*author's closure by letter*).—Referring to Mr. Shaw's criticism, we cast the various sized bars from the same ladle of iron merely to show the variation is strength in these several sections. The object of the paper was to bring out this variation and not to recommend or endorse any particular size of bar.

There is a good deal to be said in favor of a bar or bars to represent the metal in the casting, and I believe Committee A-3 on Cast Iron should seriously consider working in this direction.

MR. N. L. MOCHEL[2] (*presented in written form*).—In connection with a consideration of the paper by Mr. MacPherran, and that by Messrs. Rother and Mazurie, the results of an investigation along similar lines, just being completed at the South Philadelphia Works of the Westinghouse Electric and Manufacturing Co., may be of interest. Instead of using separately cast bars of varying section, comparison was made between standard arbitration test bars and a casting of more or less symmetrical shape, with sections varying from ½ to 4 in. in thickness.

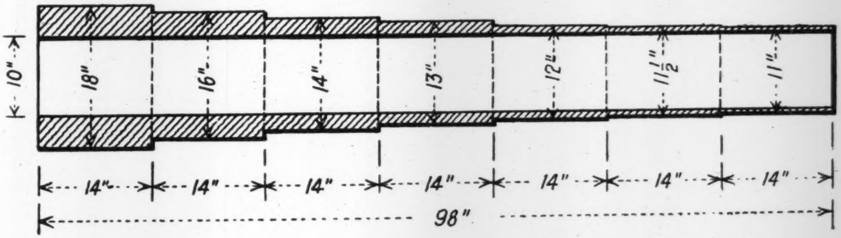

FIG. 1.—Cast-Iron Test Casting.

The much-used arbitration test bar for cast iron as described in the Society's Tentative Specifications for the Arbitration Test Bar and Tension Test Specimen for Cast Iron (A 124 - 28 T) is intended to represent the metal in the ladle and not the metal in the casting. As such, it has a definite value to the foundryman in providing a measure of the quality of the iron melted. But, as the values obtained from the arbitration test bar do not represent the castings, they cannot be safely used by the designer of equipment for important applications without some knowledge of the values to be expected in the actual casting. To provide some data as to the comparative values of arbitration test bars and varying thicknesses of metal in an actual casting, the casting shown in Fig. 1 was prepared and cut up for test purposes.

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[1] Chief Chemist, Allis-Chalmers Manufacturing Co., West Allis, Wis.

[2] Metallurgical Engineer, Westinghouse Electric and Manufacturing Co., Philadelphia, Pa.

The molds for the casting and arbitration bars were rammed with the sand slinger under normal foundry conditions and with the regular mixture of Millville gravel and old foundry sand.

The iron was made up as follows:

Malleable Bessemer Pig Iron	225 lb.
No. 3 Pig Iron	600 lb.
Iron Scrap	700 lb.
Steel Scrap	400 lb.
Silicon Iron	75 lb.

Melting was carried out in a 60-in. cupola.

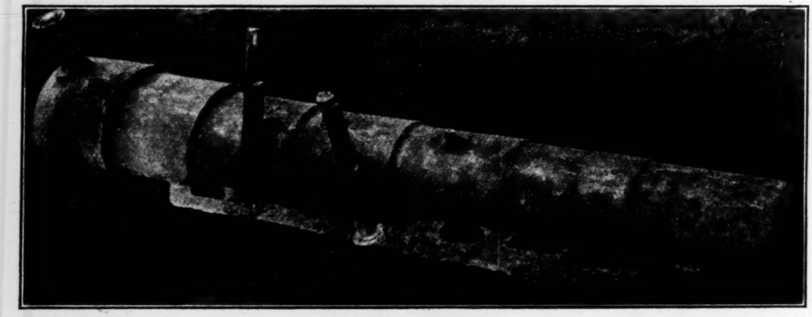

FIG. 2.—Shows Casting as Shaken Out of the Mold, with Riser and Gates in Position.

Three of the old 12-in. arbitration bars and three of the newer 18-in. bars were poured directly before the pouring of the casting, and three of each directly after the pouring of the casting. The test bars and casting were poured at a temperature of 2520 to 2500° F.

Figure 2 shows the casting as shaken out of the mold, with riser and gates in position. The casting was poured in the horizontal position as shown. Two of the arbitration bars are shown resting on the casting.

Transverse tests were made on the short and long bars on 12-in. and 18-in. supports. The long specimens were of standard 1.20-in. diameter. The short specimens were 1.25 in. in diameter, and the results were corrected to 1.20-in. diameter. The results are given in Table II.

Chemical analyses were made on one of the bars poured before the casting proper and on one of the bars poured after the casting, the results being given in Table I.

The Brinell hardness was determined towards the top and bottom of each bar. Results are given in Table II.

TABLE I.—CHEMICAL COMPOSITION.

	Silicon, per cent	Sulfur, per cent	Manganese, per cent	Phosphorus, per cent	Graphitic Carbon, per cent	Combined Carbon, per cent	Total Carbon, per cent	Nickel, per cent	Chromium, per cent
Arbitration Bar—Before	1.20	0.122	0.61	0.198	2.77	0.72	3.49	nil	nil
Arbitration Bar—After	1.21	0.122	0.61	0.200	2.77	0.72	3.49	nil	nil
A5	1.21	0.112	0.65	0.198	2.80	0.70	3.50	nil	nil
B5	1.22	0.110	0.65	0.198	2.80	0.69	3.49	nil	nil
C5	1.22	0.114	0.67	0.209	2.79	0.69	3.48	nil	nil
D5	1.23	0.114	0.67	0.198	2.79	0.69	3.48	nil	nil
E5	1.23	0.115	0.66	0.198	2.80	0.68	3.48	nil	nil
F5	1.23	0.112	0.67	0.207	2.81	0.67	3.48	nil	nil
G5	1.22	0.114	0.66	0.195	2.82	0.67	3.49	nil	nil

TABLE II.—RESULTS OF TESTS ON ARBITRATION BARS.

	Transverse Tests		Brinell Hardness		Tensile Strength, lb. per sq. in.	
	Breaking Load, lb.	Deflection, in.	Top	Bottom	Top	Bottom
18-IN. BARS—BEFORE CASTING						
No. B-1-18	2375	0.30	223	228	33 600	36 000
No. B-2-18	2420	0.30	207	207	34 400	34 600
No. B-3-18	2400	0.28	223	223		
Average	2398	0.29	218	219		
18-IN. BARS—AFTER CASTING						
No. A-1-18	2345	0.28	228	228	34 600	36 000
No. A-2-18	2525	0.28	217	228	35 000	34 800
No. A-3-18	2200	0.26	217	228		
Average	2356	0.27	220	228		
12-IN. BARS—BEFORE CASTING						
No. B-1-12	3090	0.14	223	217	35 000	34 800
No. B-2-12	2800	0.13	217	217	35 800	34 000
No. B-3-12	3500	0.15	207	217		
Average	3130	0.14	216	217		
12-IN. BARS—AFTER CASTING						
No. A-1-12	3600	0.14	228	223	34 900	35 400
No. A-2-12	3150	0.13	217	217	36 000	34 600
No. A-3-12	3400	0.15	228	223		
Average	3383	0.14	224	221		
				Average	34 911	35 025

Tension tests were made from both upper and lower ends of the broken bars, using the tension test specimen recommended in the Society's Tentative Specifications A 124 – 28 T. The results are given in Table II.

TABLE III.—RESULTS OF TESTS ON CASTINGS.

Serial	Transverse Tests		Brinell Hardness		Tensile Strength, lb. per sq. in.	
	Load, lb.	Deflection, in.	"Small" End	"Large" End	"Small" End	"Large" End
A-1	4070	0.41	187	170	29 000	......
A-2	4100	0.39	187	179	......	30 500
A-3	[a]	[a]	187	187	33 000	......
A-4	4000	0.40	179	170	......	28 000
A-5	3710	0.32	179	179	33 000	......
A-6	4430	0.39	179	179	......	[b]
A-7	4570	0.37	179	187	31 500	......
A-8	4070	0.41	179	163	......	30 000
Average	4135	0.387	182	176	30 700	
B-1	4300	0.297	170	170	28 000	......
B-2	4000	0.320	163	170	......	27 500
B-3	3200	0.255	179	187	27 000	......
B-4	3300	0.355	170	170	......	25 550
B-5	[a]	[a]	187	187	29 000	......
B-6	2500	0.360	170	170	......	29 000
B-7	3500	0.264	170	170	27 000	......
B-8	3500	0.300	170	170	......	26 500
Average	3470	0.307	172	174	27 440	
C-1	3140	0.250	170	163	25 500	......
C-2	3330	0.260	170	163	......	25 500
C-3	3020	0.210	163	156	25 500	......
C-4	3250	0.250	159	156	......	24 250
C-5	3150	0.240	163	163	24 500	......
C-6	3080	0.245	166	163	......	25 000
C-7	2500	0.220	159	163	24 500	......
C-8	3070	0.255	163	156	......	24 250
Average	3060	0.241	164	160	24 875	
D-1	2860	0.190	163	163	23 200	......
D-2	2850	0.175	163	163	......	24 800
D-3	2900	0.180	163	163	25 400	......
D-4	2870	0.170	163	163	......	24 800
D-5	2800	0.178	163	163	24 500	......
D-6	2900	0.185	163	163	......	24 600
D-7	3000	0.185	163	163	24 200	......
D-8	2900	0.180	163	163	......	24 200
Average	2885	0.180	163	163	24 460	
E-1	2810	0.180	163	159	24 000	......
E-2	2930	0.190	163	163	......	24 000
E-3	2930	0.185	166	166	24 600	......
E-4	2770	0.170	166	166	......	24 000
E-5	2830	0.180	163	163	24 000	......
E-6	2900	0.200	163	163	......	24 600
E-7	2800	0.198	163	166	23 200	......
E-8	2800	0.180	163	163	......	24 000
Average	2840	0.185	163	163	24 050	
F-1	2660	0.190	156	156	21 400	......
F-2	2760	0.175	156	156	......	23 000
F-3	2570	0.180	163	163	22 600	......
F-4	2635	0.160	156	156	......	23 600
F-5	2870	0.175	163	163	24 600	......
F-6	2860	0.175	156	156	......	24 200
F-7	2640	0.170	156	156	20 400	......
F-8	2560	0.170	156	156	......	22 000
Average	2690	0.174	157	157	22 725	

[a] Broke in machining.
[b] Spoiled.

TABLE III.—RESULTS OF TESTS ON CASTINGS (*Continued*).

Serial	Transverse Tests		Brinell Hardness		Tensile Strength, lb. per sq. in.	
	Load, lb.	Deflection, in.	"Small" End	"Large" End	"Small" End	"Large" End
G-1—Outside....	2500	0.165	156	143	24 400	......
G-2— "	2500	0.160	153	153	......	20 000
G-3— "	2650	0.160	163	156	20 900	......
G-4— "	2900	0.180	153	153	......	20 000
G-5— "	2700	0.150	163	156	23 000	......
G-6— "	2750	0.172	153	156	......	20 400
G-7— "	2620	0.165	163	156	23 400	......
G-8— "	2825	0.185	153	166	......	20 000
Average.......	2680	0.167	157	154	21 500	
G-1—Center.....	2500	0.170	149	149	23 600	......
G-2— "	1800	0.170	163	146	......	20 200
G-3— "	2900	0.175	159	153	20 200	......
G-4— "	2800	0.160	163	149	......	20 900
G-5— "	2600	0.170	163	163	24 200	......
G-6— "	2900	0.165	149	149	......	24 200
G-7— "	2800	0.175	163	156	20 000	......
G-8— "	2670	0.160	156	156	......	20 200
Average.......	2620	0.168	158	152	21 680	

The casting was parted in the lathe, at each shoulder, thus making seven sleeves each 14 in. long, and ranging in thickness as indicated in Fig. 1. These were stamped from "A" to "G" in order from the ½-in. to the 4-in. thickness. They were laid out to cut eight pieces 45 deg. apart, No. 1 being at the extreme upper point of the casting as poured, and numbering from No. 1 to No. 8 in a clockwise direction, while standing at the thinner end of the casting as a whole, and looking towards the heavier end.

These eight pieces from each sleeve were turned to a round straight bar for transverse tests. For sleeves "A" to "F," inclusive, the bars were turned with their axis midway in the wall thickness. In the case of sleeve "G," 4 in. thick, one set of bars was turned from the extreme outer portion, and one set at mid-thickness. For sleeves "A," "B" and "C" the bars were turned to 0.496, 0.625 and 0.875-in. diameter, respectively. The other bars were turned to standard 1.20-in. diameter. Transverse tests were made on each on 12-in. supports, and results corrected to 1.20-in. diameter. The results are given in Table III.

Brinell hardness readings were taken on each, at each extreme end. The "small" and "large" end of the casting is marked in each case; for example on sleeve "B," "small" end means the end of the test bar next to the sleeve "A" and "large" end, the end next to sleeve "C." Results are given in Table III.

Tension tests were made from each bar: on pieces 1, 3, 5 and 7 from the end of the broken bar next to the "smaller" sleeve, and on pieces 2, 4, 6 and 8 from the end of the broken bar next to the "larger" sleeve. The specimens used for sleeves "D," "E," "F" and "G" were in accordance with the recommendations of the Society's Tentative Specifications A 124 – 28 T. The specimens from sleeves "A" and "B" were 0.357 in. in diameter and those from sleeve "C" were 0.505 in. in diameter, produced with the same form tool that is used to produce the standard specimens. Results are given in Table III.

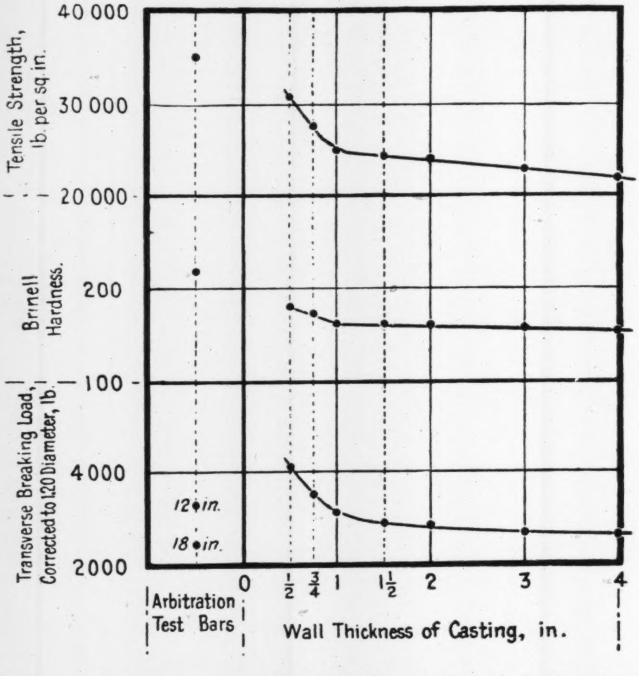

FIG. 3.—Curves Showing Average Physical Properties of Arbitration Bars and Actual Casting of Varying Wall Thickness.

Chemical analyses were made from specimens No. 5 from all seven sleeves. Results are given in Table I.

The average values have been plotted in Fig. 3. It will be noted that the arbitration bars show greater hardness and tensile strength than the thinner sections of the casting, but that in the matter of transverse test, the ½ and ¾-in. sections give higher values than the arbitration bars.

It will be noted that as the thickness of the casting increases, there is a marked falling off in properties up to 1-in. thickness but a more gradual falling off as the thickness is increased to 4 in.

The uniformity of composition throughout the casting is of interest.

Strictly speaking, the results reported and the comparison of the arbitration bars and casting proper are representative of but one casting and lot of test bars poured from a given heat of iron of the type shown by the composition. It is believed, however, that the data are of value to the designer, as the casting and test bars were prepared under normal controlled conditions, and that other castings and bars prepared under similar conditions may be expected to behave in somewhat like manner, and that the results indicate what may be expected in varying thicknesses of metal of this particular grade.

The writer wishes to acknowledge the assistance of Messrs. Kuebler, Murray and Place in preparing the material and carrying out the tests.

MR. J. G. PEARCE[1] (*by letter*).—It appears that there is a slight error on page 85 of the paper by Messrs. Rother and Mazurie where the text says, "For comparing sections of the same size and shape, the modulus of rupture is a good criterion. For comparing sections of different sizes it is not particularly applicable unless some account is taken of the differences in size." I find that the modulus of rupture is a suitable criterion for comparing sections of the *same shape* and of *different sizes*, but that on account of differences in stress distribution across the bar it does not afford a ready method of comparing different *shapes*. As their own results have proved, a square bar gives lower modulus figures than a round bar of the same metal.

MESSRS. W. H. ROTHER[2] and V. M. MAZURIE[3] (*by letter*).—In preparing our paper we attempted to cover the subject thoroughly, but briefly. The statement to which Mr. Pearce refers, evidently does not convey the thought we wished to express.

The idea we wished to bring out was that the modulus of rupture computed from the test of a small bar would not be the same as that computed from the test of a larger size bar having the same shape. The results of the tests on the different size bars would not be comparable figure for figure. However, a good comparison is obtained if account is taken of the regular relationship existing between the modulus of rupture for a small bar and that of a large bar.

MR. J. S. VANICK.[4]—I should like to direct attention in Mr. Bolton's paper to the volume-area relation, which again Mr. Mochel has illustrated in the step test bar, particularly with reference to large castings—castings weighing say over 200 lb. The volume-area relation is perhaps as good a first hand one as could be devised, and Mr. Bolton and others who have worked upon it should be congratulated over that derivation. However, from the practical foundryman's standpoint, it is not a very easy thing to run a casting in such a way that the cooling rate through a section is uniform. It would have to be run something like a pipe or cylinder and top poured, and that is very rarely the case. Most castings are bottom-gated to keep them clean, and then there are cores present, and the combination of the position of the cores and the position of the gate means that this volume-area relation will be distorted to some extent, particularly in that section of the casting close to the gate. One can readily see that if you have a four-ton casting or even a 600-lb. casting with a single gate bringing in all the hot metal down at the bottom (say a cylinder being gradually filled) the sand mold at the bottom will be considerably preheated. In a small casting, the temperature effect there is not so great. So that, from the engineering standpoint, I believe that the best approximation of what the strength of a casting would be, would be to add some increment to the volume-area relation, and in that way approximate the cooling condition in the preheated portion of the casting. This would mean that if the volume-area relation indicated that a 3-in. diameter bar should be used, it would be better in the case of a 600-lb. or a 1000-lb. casting to use a 4-in. bar to compensate for the preheating of the mold near the gate; or perhaps the best way of all would be to actually determine the cooling rate in the section of the casting that the engineer is most interested in, and then fit the test bar dimensions to that rate.

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[1] Director, The British Cast Iron Research Assn., Inc., Birmingham, England.

[2] Metallurgist, Buffalo Foundry and Machine Co., Buffalo, N. Y.

[3] Assistant Metallurgist, Buffalo Foundry and Machine Co., Buffalo, N. Y.

[4] Research Metallurgist, Elizabeth, N. J.

MR. J. E. HURST[1] (*presented in written form*).—Referring to Mr. MacKenzie's paper on the elastic properties of cast iron, these elastic properties are of great importance in many industrial applications. In cast-iron piston rings or spring rings it is obvious that the elastic properties of the material are of primary importance.

This has been realized in England and the British Engineering Standards Association in their specifications No. 5004 and 4K6 for automobile and aircraft piston ring material, respectively, have embodied a test designed to give some indication of the elastic properties.

In view of the fact that the stress-strain ratio for cast iron is not constant, it has been considered desirable to differentiate the stress-strain ratio obtained in this test from the ordinary Young's modulus of elasticity generally denoted by the symbol *E*. The symbol E_n is therefore used and the elastic value obtained by this test is commonly referred to as the E_n value. The value is obtained under the carefully designed conditions of the test by means of the formula given below, the test being carried out on a slit ring. A test ring is machined from the piston ring drum casting to a thickness of not less than $\frac{\text{Diameter (uncut)}}{34}$ and a piece cut out of the ring so as to leave a free gap of not less than $2.75t$ and not more than $3t$. The width, b, and the radial thickness, t, shall be determined and a diametral load, Q, in pounds sufficient to close the gap to less than $0.25t$ shall be applied. The change in gap, δ, and the mean external diameter of the closed ring, d, shall be measured and shall be such that E_n when calculated from the following formula is not less than 15,500,000 lb. per sq. in.:

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[1] Newton, Chambers and Co., Limited, Thorncliff Iron Works, Sheffield, England.

$$E_n = \frac{5.37\left(\frac{d}{t} - 1\right)^3 \times Q}{b\,\delta}$$

This test is used extensively in the manufacture of material for piston rings.

There are two points in Mr. MacKenzie's paper that have attracted my attention and to which, from my experience with the above test, I should hesitate to agree. Mr. MacKenzie concludes that the modulus of elasticity is an excellent index of the resistance to plastic deformation, which latter is identical presumably with what we call permanent set. My experience shows that it is possible to have iron of uniformly high elastic properties showing widely different values of permanent set and *vice versa.*

The second point is the statement that the modulus of elasticity is very closely related to the analysis when the cooling conditions are uniform. This statement is much too broad. We ourselves find the utmost difficulty in tracing variations in the E_n value to the chemical composition. By far the most important features which affect the E_n value appear to be the quality of the pig iron used, whether or not steel is used in the mixture and its quantity. These and similar questions are extremely difficult to investigate quantitatively.

I hope that Mr. MacKenzie's paper will do much to stimulate further investigation into the elastic properties of cast iron.

(By letter).—Since presenting my written contribution on Mr. MacKenzie's paper I have had the pleasure of a personal conversation with Mr. MacKenzie on the subject of the modulus of elasticity of

cast iron. It appears to me advisable as a result of this conversation to amplify my remarks.

The most important conclusion in Mr. MacKenzie's paper is taken apparently from his previous paper entitled "The Influence of Phosphorus on Cast Iron."[1] This conclusion is contained in the following statement abstracted from Mr. MacKenzie's paper: "that it [the modulus of elasticity] is very closely related to the analysis when cooling conditions are uniform."

I have had the opportunity of studying the previous paper referred to above, containing the main evidences on which this statement is based. I venture to suggest in the first place that the evidence in this paper does *not* substantiate the conclusion given above. The evidence shows that the modulus of elasticity is *only broadly* related to the sum of the total carbon contents plus one third of the silicon contents. The limits of variation are so wide that even this broad relation is obviously of little value in attempting to control the modulus of elasticity.

In my experience of the control of the modulus of elasticity of cast iron for piston rings I find it impossible to say that this property is very closely related to the chemical composition when the cooling conditions are uniform. I have selected from the many results at my disposal three sets of results which illustrate three types of experience which I have met. These results are plotted in the accompanying Fig. 4 of Mr. MacKenzie's closure.

In each of these three sets, the size of the castings and the test specimens are identical and the cooling conditions are carefully controlled. In the first set tests were made on three rings of identical dimensions machined from the same casting. These results illustrate a variation in modulus of elasticity in the same casting. Incidentally this variation is large enough to reject the material according to the main specifications to which we have to work.

The second set represents results taken from a series over a period when a deliberate reduction in total carbon content was made in anticipation of an increase in E_n value. Here again the size of the castings and test specimens were identical, and it will be seen that the anticipation of an increase in modulus by lowering the total carbon content was not realized.

In the third set, as in the previous cases, the specimens were of identical sizes machined from identical castings. The E_n value varies from 12,920,000 to 17,920,000 lb. per sq. in. The irregular behavior of the tension tests which in three cases are really a form of modulus of rupture will be noticed also.

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[1] *Transactions*, Am. Foundrymen's Assn., p. 988 (1926).

These results are all determined according to the method laid down in British Engineering Standards Association Specification for Aircraft Material 4K6. There are of course quite definite reasons apart from the chemical composition and the cooling conditions to which the variation above mentioned is to be attributed.

MR. IHEI SUGIMURA[1] (*presented in written form*).—I wish to congratulate Mr. MacKenzie on his valuable work, and to express my appreciation of the reference he has made in his paper to my paper which appeared three years ago in the *Journal* of the Society of Mechanical Engineers in Japan.

Having appeared in the "Far East," I am somewhat astonished that it should have received such notice.

I understand Mr. MacKenzie regards "modulus of elasticity" of most importance in the consideration of elastic properties of cast iron. The general meaning of "modulus of elasticity," as the author remarks, is the quotient of elastic stress intensity in a longitudinal direction (tension or compression) divided by the corresponding longitudinal strain induced by that stress and is commonly considered as constant within the elastic range. In such a metal as cast iron, the above quotient, as is well known, is by no means constant, and so exponential or hyperbolic functions must be employed for the expression of the relation of stress and strain. In other words, modulus of elasticity of cast iron changes its value at each intensity of stress.

Moreover, in the establishment of the usual formulas employed for beam calculations, stress and strain are generally supposed, as one of several assumptions, to have straight line relations with each other. Therefore, when we apply these formulas to cast-iron beams, considerable diversity between calculated and actual results is to be anticipated.

Such being the case, I am inclined to think that, for elastic study of cast iron, to use so vague a value as modulus of elasticity as the basis of consideration is unscientific, and further, may often lead to misunderstanding, although a cast-iron beam having definite form, size and span may practically, to some extent, be used for a test specimen in comparative study.

Again, to use the words "modulus of elasticity" outside the range of elasticity, for example, "modulus of elasticity at breaking point" (shown as abcissas in Fig. 3) is rather misleading, because the values do not indicate any value expressing elasticity. That is, they do not express any value regarding what amount of strain returned to its original length, but they show stress due to relative strain (in which a fair amount of plastic strain is included).

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[1] Sanki Engineering Co., Ltd., Uchisaiwai-cho Kojimachi-ku, Tokio, Japan.

Aside from the nomenclature I admire the author's idea of arranging graphite content, modulus of rupture, Brinell hardness, etc., against the stress-strain quotient (author's modulus of elasticity) for the study of cast iron, because by this quotient the properties of cast iron mentioned are well arranged, and further I quite agree with Mr. MacKenzie's view concerning "set" of cast iron, which he attributes to crushing of graphite flakes or to squeezing of iron grains into the flakes or the voids surrounding them. I should wish to ask the author, however, if he has made any microscopic study to scientifically prove this theory. If not, I hope he will try and show us how the matter develops.

MR. J. W. BOLTON[1] (*by letter*).—In his very interesting paper Mr. MacKenzie has arrived at some conclusions which do not seem quite fully established by his experimental data. Permanent set or deformation in gray iron after initial stress is a phenomenon clearly recognized for many years. Mr. MacKenzie explains this set as due to crushing of the graphite flakes and claims that this is proportional to the amount of graphite. This hypothesis may be proved later by different methods of experimentation but does not seem fully proved by the data presented.

Mr. MacKenzie states that the restressed bar was loaded to three-fourths or 75 per cent of its ultimate breaking load as determined in the transverse test. In the transverse test two factors prevent accurate determination of the proportionate ultimate fiber stresses (inferred as 75 per cent in the paper) or the actual stress intensities. The calculation of modulus of rupture in cast iron is empirical, in that this material does not comply with the fundamental laws governing application of beam formulas. For example, cast iron varies in tensile and compressive stress, so that the position of neutral stress axis, for a given iron, varies with each increment of load. Furthermore, for different irons this displacement varies with a given load. Thus the irons examined by Mr. MacKenzie were not proportionately stressed, nor can the extreme fiber stresses be even given.

Further, in a series of tension tests (where proportionate loads can be approached) we find on restressing beyond a certain amount the development of an ultimate stress-strain curve distinctly different from that on initial stress, or that obtained at lower restressing loads. For example, at a 75-per-cent proportional restressing for 25 times and rerunning to break, we get almost exact proportionality of stress to strain up to about 80 per cent of the ultimate strength. Then there is a very sharp knee. The curve is not at all like those called for in the hyperbolic formulas of Sugimura. Also there is a continued, although diminished, permanent set for this iron at each stressing up to and including 25 stresses. However, when we take the same iron and restress it to 55 per cent of its ultimate tensile strength we get permanent deformation on the first few stresses only and after 25 restressings the ultimate stress-strain curve remains of the hyperbolic formula type. Thus the stress intensity is of extreme importance in that below and above certain limiting stresses (different for each grade of iron) different actions take place: that is, first a mere compacting of graphite, latterly an actual distortion and possibly incipient fracture within the matrix. The 75-per-cent point, chosen by MacKenzie, is unfortunate for (assuming it is 75 per cent of ultimate fiber stress, although we cannot evaluate that in a transverse test) he is in a critical range. Thus the results cannot prove his contention.

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[1] Chief Chemist and Metallurgist, The Lunkenheimer Co., Cincinnati, Ohio.

As a comparison, take a mild and a cold-worked steel of respective elastic limits of 45 per cent and 70 per cent, stress each to the 60-per-cent point. There will be much deformation in the soft steel, practically none in the cold-worked steel—and neither has graphite.

It so happens that we are conducting a rather extensive investigation of the elastic properties of gray iron, yet the work is so voluminous we cannot condense it sufficiently to show wherein on other grounds the paper discussed apparently disregards certain fundamental factors.

MR. J. T. MACKENZIE (*author's closure by letter*).—I should like to call Mr. Bolton's attention to the statement at the bottom of page 98, that "the work already done indicates the need of a careful investigation along these lines with accurate apparatus and with due regard to the fatigue limit in repeated bending."

It was certainly not my idea that I had settled all of the points of interest in the elastic properties of cast iron. However, as to the objection to the graphite flake explanation, if the bar is stressed above its elastic limit (a point which I do not accept), why does it not flow on each successive application of the load as steel does? If Mr. Bolton means to imply that cold work is responsible, why is the bar less stiff than originally if loaded in the reverse direction? For instance: one bar of set No. 797 was loaded to 1500 lb. (fiber stress 27,000 lb. per sq. in.) and deflections read. The results were as follows:

	Loading	Deflection, in.
Side *A* in tension	No. 1	0.375
	No. 2	0.28
	No. 3	0.285
	No. 4	0.28
	No. 5	0.275
	No. 6	0.28
	No. 7	0.275
Side *B* in tension	No. 8	0.445
	No. 9	0.295
	No. 10	0.29
	No. 11	0.28
	No. 12	0.28
	No. 13	0.28
	No. 14	0.285
Side *A* in tension	No. 15	0.44
	No. 16	0.30
	No. 17	0.295
Side *B* in tension	No. 18	0.43
	No. 19	0.30
	No. 20	0.29
Side *A* in tension	No. 21	0.44

A further test on this point was made on an iron similar to No. 829. The curve had a "knee" at about 1200 lb., which might be an indication of an elastic limit. One bar was loaded twenty-one times to 2000 lb. and another bar twenty-one times to 1000 lb., after which each was carried to failure (the twenty-second loading). Careful profile readings were made on the bars before loading and after the first, second, third, seventh and twenty-first bendings. The comparison of actual "set" is shown below:

SET NO. 1086

Loading	Deflection, in. At 2000 lb.	Loss of Deflection, in.	Set by Gages, in.
No. 1	0.272	.....	.....
No. 2	0.243	0.029	0.019
No. 3	0.244	0.028	0.020
No. 7	0.244	0.028	0.023
No. 21	0.246	0.026	0.024
No. 22	0.240	0.032	0.028

LOADING	DEFLECTION, IN.	LOSS OF DEFLECTION, IN.	SET BY GAGES, IN.
	AT 1000 LB.		
No. 1	0.111	.....	.....
No. 2	0.108	0.003	0.003
No. 3	0.107	0.004	0.005
No. 7	0.102	0.009	0.005
No. 21	0.103	0.008	0.005
No. 22	0.103	0.008	0.005

As the results in the paper were based on loss of deflection after seven loadings the error would seem rather insignificant.

Another test was made on a considerably softer iron at loadings of 1500, 1000 and 500 lb., and in this the curve had a pronounced "knee" at 750 lb. and another rather sharp one at 1500 lb. The same readings were taken, and the comparison between loss of deflection and set by the gages was very satisfactory, as is shown below:

SET NO. 1120

LOADING	DEFLECTION, IN.	LOSS OF DEFLECTION, IN.	SET BY GAGES, IN
	AT 1500 LB.		
No. 1	0.287	.....	.....
No. 2	0.26	0.027	0.026
No. 3	0.247	0.040	0.036
No. 4	0.246	0.041	0.041
No. 7	0.246	0.041	0.043
No. 21	0.243	0.044	0.044
	AT 1000 LB.		
No. 1	0.184	.....	.....
No. 2	0.165	0.019	0.016
No. 3	0.166	0.018	0.020
No. 7	0.161	0.023	0.022
No. 21	0.162	0.022	0.022
	AT 500 LB.		
No. 1	0.077	.....	.....
No. 2	0.073	0.004	0.003
No. 3	0.071	0.006	0.003
No. 7	0.071	0.006	0.003
No. 21	0.071	0.006	0.007

In set No. 1086 the bar which had been tested to 1000 lb. gave practically no evidence of its previous treatment when the final bending curve was obtained (2570 lb.). The bar tested to 2000 lb. did give a sharp "knee" at about 2100 lb. It also broke at 2570 lb. A reference to the original curve, however, shows that all three bars bent 0.11 in. in the final 500 lb. of the breaking curve, showing that as soon as the load exceeded that of the previous stressing the bar behaved as the original material.

In set No. 1120, the soft iron, a change of curvature was noted in each bar as it passed the point where it had been previously stressed. The fact that it nearly resumed the original curvature is shown by the following figures:

DEFLECTION BETWEEN 500 AND 1500 LB.

Bar previously loaded twenty-one times to 500 lb.	0.222 in.
Bar previously unstressed	0.221 in.

DEFLECTION BETWEEN 1000 AND 1900 LB.

Bar previously loaded twenty-one times to 1000 lb.	0.285 in.
Bar previously unstressed	0.285 in.

DEFLECTION BETWEEN 1500 AND 1700 LB.

Bar previously stressed twenty-one times to 1500 lb.	0.065 in.
Bar previously unstressed	0.065 in.

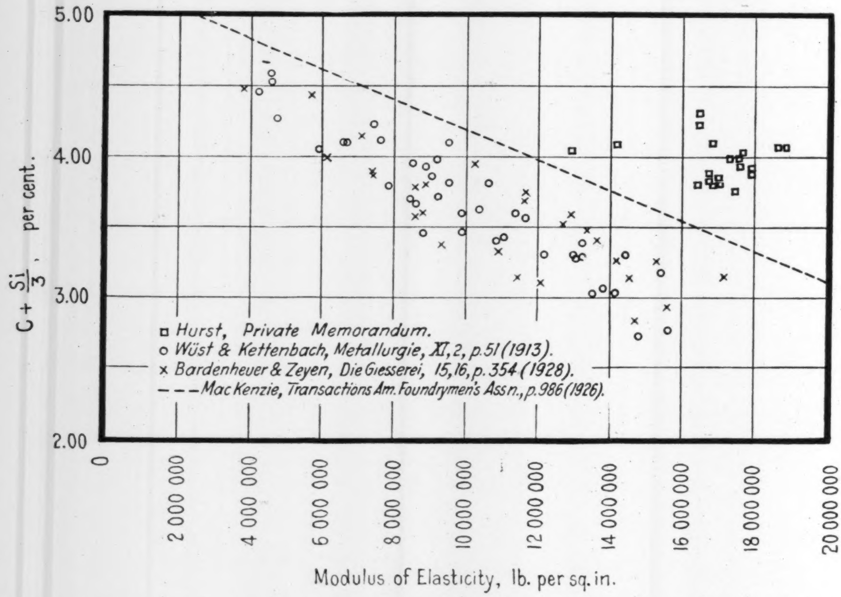

FIG. 4.—Showing Relation of Modulus of Elasticity to Carbon Plus One-third Silicon.

I can see nothing in these data to confute the graphite flake idea or to indicate an elastic limit. Though, undoubtedly, from the work of Thum and Ude,[1] who succeeded in duplicating the stress-strain curve for cast iron by putting slits and notches in steel specimens, there must be a point at which the elastic limit of the matrix would come into play, this is probably very high, for in ordinary pearlitic cast iron one would expect the elastic limit of the matrix to be rather above the strength of the material, or close to it, and on account of the grooved and notched effect of the graphite flakes it must break as a brittle material, and even a steel would break so with the necessary discontinuity of section to duplicate cast iron.

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[1] A. Thum and H. Ude, *Die Giesserei*, May 31, 1929.

I fully agree with Mr. Bolton that the fiber stress used (really modulus of rupture when calculated from the breaking load) is empirical and varies with the load and the iron—the higher the load or softer the iron the greater the error—but as it is used as a percentage of the ultimate it could not be a large factor in the calculations given in the paper.

Replying to Mr. Hurst's doubts as to the relation between modulus of elasticity and analysis, I have plotted in the accompanying Fig. 4 the results of Wüst and von Kettenbach[1] on low-phosphorus iron with large variations in both silicon and total carbon, and the results of later work by Bardenheuer and Zeyen,[2] using all of their low-phosphorus bars melted and poured at normal temperatures. The average line from my paper on "Phosphorus in Cast Iron"[3] is drawn in, and some results on piston rings referred to by Mr. Hurst are also indicated. The spread at the lower end for the German bars at a value of carbon plus one-third silicon of 3.10 per cent is almost exactly the same as that in the piston rings at 4.10 carbon plus one-third silicon, but I am sure that no one would deny the very evident slope of the line for the German bars. As the piston rings are cooled very rapidly and the modulus is not the ultimate they should lie much higher on the graph. The high average phosphorus of my bars (about 1 per cent) would naturally shift them above the German bars which were mostly 0.05 per cent. As Mr. Hurst states, "The modulus of elasticity is only broadly related to the carbon plus one-third of the silicon," but can we do better than pursue a broad relationship with the hope of ultimately establishing one more definitive?

I thank Mr. Sugimura for his comments and trust that we may be able to find something definite by a microscopic examination which is progressing rather slowly and is so far inconclusive.

MR. I. J. FAIRCHILD.[4]—There is one point in reference to cast iron that has not been discussed here. Mr. MacKenzie has been advocating for some time the use of an energy absorption computation to determine the value of cast iron, particularly in reference to pipes. I think it would be well to study this idea of Mr. MacKenzie's.

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[1] F. Wüst and W. von Kettenbach, *Ferrum*. Vol. XI, p. 51 (1913–1914).

[2] P. Bardenheuer and K. L. Zeyen, *Die Giesserei*, April 20, 1928.

[3] J. T. MacKenzie, "Influence of Phosphorus on Cast Iron," *Transactions*, Am. Foundrymen's Assn., p. 988 (1926).

[4] Mechanical Engineer, U. S. Bureau of Standards, Washington, D. C.

Neither strength nor the secant modulus are in themselves a full measure of the quality of cast iron. Certainly when one is considering pipe in connection with external loading, the impact test is not the final criterion, and I should like to add my personal plea to Mr. MacKenzie's recommendation, that Committee A-3 on Cast Iron consider his method of computing energy absorption or resilience.

MR. MACKENZIE.—There is one thought I should like to bring out; we cast iron people have had this unreliability of cast iron rammed down out necks so much that at one time we almost believed it ourselves, but the engineers and purchasing agents have had as much to do with that evil repute as the manufacturers of cast iron. A man who is making good cast iron for the purpose for which it is intended and is spending the necessary money to make that cast iron and make it well finds himself underbid in the purchasing agent's office by a man who does not know what it takes to make good cast iron and is not prepared to make it, will not spend the money on it. In a short time after this fellow has taken the business away from the good foundry, you find the pattern gone to the malleable foundry or the steel foundry or somewhere else. In a few years it probably comes back to the original founder, who, by that time, is in the care of the sheriff; and I should like to leave that thought with you, that the engineer has not appreciated the fact that the man who is making cast iron in his backyard cannot make it as well as the man who is equipped to do it properly, and I think we should do more thinking about the quality and grade of cast iron and less about the price of it.

In England I saw them making grate bars out of perlit iron, which has an analysis of 1.00 per cent silicon and 3 per cent carbon, for which they were charging fourteen cents a pound. Engineering concerns have found it economical to pay for these low-silicon castings in preference to buying grate bars at four cents a pound made in the ordinary manner.

MR. BOLTON.—I wonder how many cast iron foundrymen, those interested in extending the uses of cast iron, realize that its use in engineering service oftentimes is limited to 450° F., and that this condition has arisen largely through the use of improper materials, that probably would have stood more than 450° F., but certainly were unsuited to 750, 800 and 900° F. to which they were exposed. It seems to me that it would be a very good move on the part of Committee A-3 on Cast Iron to give more definite consideration to the temperatures at which different grades of cast iron can be satisfactorily used. Some of them can be used satisfactorily at 700 and even 750° F. for long periods of time, and many have been, but still there is a general fear engendered by the former selection of improper grades so that all grades are limited in a great many cases to an absurdly low temperature.

MR. DAVID MACINTOSH.[1]—I should like to ask Mr. Bolton at what temperature iron will start to grow or increase in size?

MR. BOLTON.—That all depends on the type of iron you are using. Growth comes in two classifications: one growth is when you reach a range where you have an increase in graphitization; that is somewhere below the eutectoid point. Then there is another kind of growth which is simply corrosion, where you have an open-grained iron, and oxidation and other corrosion phenomena proceeds along these grains and the material gradually expands. In that case the condition is not caused by temperature but by the corroding action of the iron. Generally a very close-grained iron is pretty good when run at 700 to 750° F. I remember an inquiry referred to me four or five months ago; I do not remember the exact temperature; the inquirer had cast iron that had been in use in a steam line for twenty-four years and had given satisfactory service, and thought he had better take it out because of propaganda against danger of growth in gray iron.

MR. RICHARD MOLDENKE.[2]—I refer again to what has been said about the difficulty of getting proper results with cast iron when the foundryman makes a variety of classes of castings from the same mixture of irons in his cupola. If castings brought better prices, more care could be given the matter, and several heats run instead of but one. Then the mixtures could be adjusted to the needs of the work. It is the same situation as with the blast furnace. If foundrymen were willing to pay for better pig iron than is made to-day, they would get it.

FATIGUE PROPERTIES

MR. MOCHEL (*presented in written form*).—Referring to the paper by Mr. Kommers, a number of fatigue tests on cast iron have been made at the South Philadelphia Works of the Westinghouse Electric and Manufacturing Co. as a part of a general investigation in the matter of welding.

The specimens were prepared from pot castings 12 in. in outside diameter, 10 in. in inside diameter and 10 in. long, by slitting in a lengthwise direction.

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[1] Sacks-Barlow Foundries, Inc., Newark, N. J.

[2] Consulting Metallurgist, Watchung, N. J.

The composition was as follows:

Silicon, per cent	1.33
Sulfur, per cent	0.106
Manganese, per cent	0.66
Phosphorus, per cent	0.282
Graphitic Carbon, per cent	2.72
Combined Carbon, per cent	0.74
Total Carbon, per cent	3.46

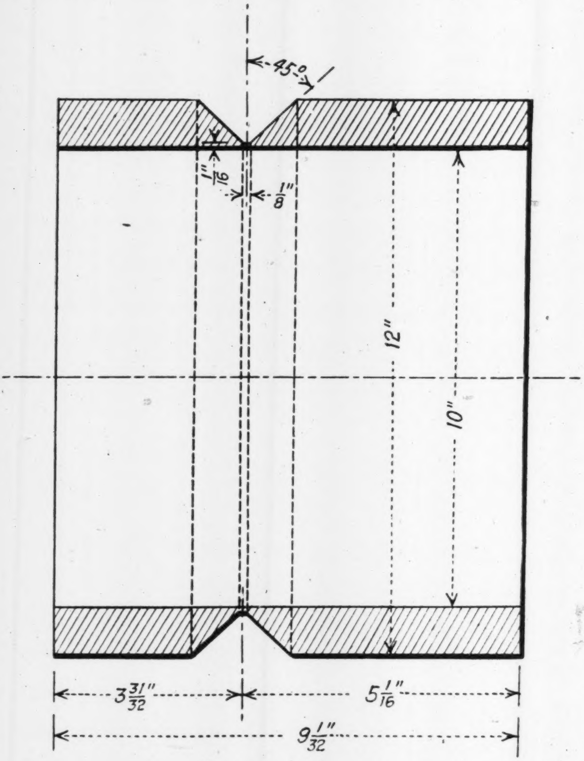

Fig. 5.—Shows Preparation of Cast-Iron and Steel Pots for Welding Tests.

The Brinell hardness, on 16 determinations, ranged from 196 to 207.

The tensile strength, 8 tests, ranged from 28,500 to 32,640 lb. per sq. in.

The Izod impact value, 8 tests, ranged from 3 to 10 ft-lb.

The endurance limit, determined on a cantilever beam specimen, was 13,500 lb. per sq. in.

The results of tests on a gas-welded pot may be of interest. The pot was one of those referred to above, from the same heat. It was prepared for welding by practically parting in two as shown in Fig. 5. The $\frac{1}{16}$-in. thickness was left merely to hold the two pieces together during the early stages of welding. The V groove was then filled by welding, using cast-iron welding rods and the oxy-acetylene flame. This recreated a full pot as before, and specimens for test were prepared as before by slitting in a lengthwise direction.

One of the pieces was ground and etched and is shown in Fig. 6. The added metal can be seen about mid-length. A very sound weld has resulted.

In machining the test specimens, they were so laid out as to bring the weld at the center of the gage length of the tension test specimens, and to cause the welded metal to extend over and to each side of the $\frac{3}{4}$-in. length of constant stress of the cantilever beam fatigue specimen.

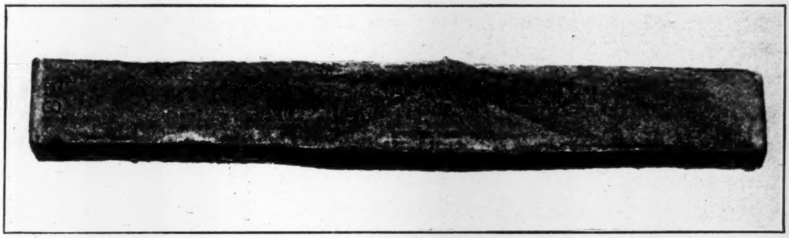

FIG. 6.—Test Specimen Ground and Etched.

The tensile strength, on 8 tests, ranged from 21,000 to 30,500 lb. per sq. in.

The endurance limit determined was 12,000 lb. per sq. in.

IMPACT TESTING

MR. SHAW (*by letter*).—Referring to the paper on impact testing by Mr. Bornstein, the writer has for many years used a machine similar to that described by Cook, but with a 35-lb. drop weight. But here again testing on one size cast bar is useless, unless there is not much variation in the mixture. For instance, with a roll mixture, the $1\frac{1}{2}$-in. bar is mottled, and broke the third drop at 7 in. But taking the 3-in. down runner from the same roll and machining it to $1\frac{1}{2}$ in. square gave a drop of 27 in. before it broke. There is also no conversion figure for the foot-pounds registered that will enable the designing engineer to check his work. The test is a useful shop test and under equal conditions will yield useful information, but that is all.

MACHINEABILITY AND WEAR

MR. H. BORNSTEIN.[1]—I think there should be considerable discussion on the two papers on wear and machineability. I think that there is more discussion in shop practice on those two items than anything else in connection with cast iron. In production work they are not so much interested in physical properties and the relation of the test bar to casting and all that sort of thing; they are vitally interested in machineability and in performance or wear. That is particularly true in the automotive industry where a good portion of the tonnage in gray cast iron is consumed.

It is strange that with all the attention that has been given to wear, there is relatively little known about the wearing qualities of cast iron. I had occasion several years ago to make an investigation of the methods employed in various automotive plants in respect to the production of automotive cylinders. I found that in some plants alloys are added to the ladle, such as nickel, chromium, molybdenum; in other cases alloys are added through the pig iron, as in the use of Mayari pig iron, but in no case did the user have absolute proof that the addition of the alloy was of considerable value or of any value to performance. It was assumed that since the alloy gave increased hardness to the casting without increasing the difficulties in machining, it was a valuable property, and there are thousands of dollars, at a very conservative estimate, spent every day on alloys in cast iron to increase wear, or the hope is that they increase wear, and I hope that somebody can give us some facts on that particular point.

In reference to machineability, every production man that you meet who is engaged in the machining of castings wants to know the factors that affect the machineability.

(*By letter*).—Several years ago, we became interested in reducing the amount of wear on tractor cylinders. These cylinders had approximately the following composition:

Silicon, per cent	2.35
Sulfur, per cent	0.08
Phosphorus, per cent	0.18
Manganese, per cent	0.65
Total Carbon, per cent	3.35

The iron was produced in the cupola and from 10 to 15 per cent of steel was added to the charge. It was desired to secure the additional resistance to wear by making alloy additions to the ladle, in view of the fact that it was not convenient to change the composition of the iron coming from the cupola. The first problem was to determine upon the suitable alloy addition to secure a material increase in Brinell hardness of the bore of the cylinder without increasing the difficulty in machining. After a large number of test runs had been made, it was found that an addition of approximately 0.60 per cent of nickel and 0.20 per cent of chromium would result in an increase of about thirty points in Brinell hardness without increasing machining difficulty. In other words, the increase in Brinell hardness was obtained without any sacrifice in machining qualities.

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[1] Director of Laboratories, Deere and Co., Moline, Ill.

The next question to be determined was whether an increase in hardness resulted in any decrease in wear. It was first proposed to attempt to watch a number of tractors in the field. Due to varying conditions in the field, it was decided that this would be a very large job, inasmuch as a very large number of tractors would have to be followed in order to secure an average result which would mean anything. The next plan was to run some tractors under full load at our experimental laboratory and very carefully determine the wear. The procedure outlined was as follows:

1. Measure cylinder block and pistons before assembly into tractor in accordance with standard checking print.
2. Assemble block and pistons into tractor on transmission brace.
   (*a*) Crank case oil—Mobile BB diluted one-third with distillate.
   (*b*) Filter collar removed from air cleaner.
   (*c*) Dirt or abrasive injection.
3. Tractor to be operated on transmission brace for a period of 20 hours in high gear.
4. Three pounds of dust to be fed in at the rate of ½ lb. per hour.
5. Tear down and measure block and pistons for wear.

The following tabulation shows the analyses of the cylinder blocks tested, and also shows the average wear on each block together with the average Brinell hardness on each block.

ALLOY IRON CYLINDERS

	No. 1	No. 3	No. 5	No. 6	No. 7
Silicon, per cent	2.38	2.35	2.38	2.44	2.38
Sulfur, per cent	0.072	0.057	0.064	0.087	0.068
Phosphorus, per cent	0.18	0.18	0.19	0.17	0.16
Manganese, per cent	0.69	0.65	0.69	0.71	0.66
Combined Carbon, per cent	0.46	0.42	0.44	0.39	0.38
Total Carbon, per cent	3.24	3.31	3.30	3.30	3.33
Nickel, per cent	0.68	0.67	0.68	0.51	0.69
Chromium, per cent	0.22	0.21	0.22	0.23	0.21
Average Wear, in.	0.00583	0.00570	0.00591	0.00578	0.00684
Average Brinell Hardness of Bore	156	152	159	145	143

PLAIN IRON CYLINDERS

	No. 2	No. 4	No. 8	No. 9
Silicon, per cent	2.30	2.34	2.35	2.34
Sulfur, per cent	0.089	0.114	0.116	0.113
Phosphorus, per cent	0.17	0.17	0.16	0.18
Manganese, per cent	0.70	0.54	0.55	0.58
Combined Carbon, per cent	0.24	0.19	0.24	0.22
Total Carbon, per cent	3.43	3.47	3.55	3.39
Average Wear, in	0.00969	0.00734	0.00715	0.00870
Average Brinell Hardness of Bore	118	130	120	123

It will be noted that the alloy iron cylinder blocks had less wear than the plain iron cylinder blocks. Furthermore, the average wear was quite uniform on the alloy iron cylinder blocks. It will further be noted that the average Brinell hardness was considerably higher on the alloy iron cylinder blocks than on the plain iron cylinder blocks. Along with this, the combined carbon content was higher on the alloy iron cylinder blocks than on the plain iron cylinder blocks.

We feel that our tests have indicated to us the value of increase in Brinell hardness in the bore and an increase in the combined carbon content, due to the addition of nickel and chromium to the ladle.

MR. MOLDENKE.—In connection with what Mr. Bornstein has said, I would add that the general situation in the foundry is not good. The foundryman wants to have machineable castings, and he wants them to wear well. Yet he usually makes both kinds of castings with the same mixture in his cupola, as he cannot hope to run a dozen different kinds of iron in the same heat. It is only in very special cases, where the foundryman has but one line of work to make, such as automobile cylinders, or piston rings, or brake shoes, and the like, that he can specialize and run the same class of metal for the entire heat. Here he can get the physical structures desired fairly well.

In Europe, where they make castings of mixtures intended to give the properties desired, irrespective of whether the machine shop complains of hardness or not, one often sees a line of cupolas in operation daily, each one melting a different mixture. There they can give better service. Here such a procedure would be too costly.

MR. F. B. COYLE.[1]—The technical files of my company contain considerable data on the effect of alloys in cast iron increasing resistance to wear. Most of these data were obtained in cooperative developments with various other companies. In most cases under these circumstances such companies prefer to have the results obtained considered confidential. Because of this fact it is not always possible to divulge specific results. Results of experience with alloy cast-iron dies for drawing sheet metal show that alloys do produce an enormous increase in life due to increased resistance to wear. In many special cases alloys will appreciably aid machineability. This is particularly true in cases where chilled corners or edges are encountered in which instance the addition of a graphitizing element such as nickel will facilitate machineability by eliminating the chill.

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[1] Research Metallurgist, International Nickel Co., Bayonne, N. J.

Mr. Moldenke's point regarding a single cupola mixture is well made. In a particular foundry it may be desirous to run a single mixture suitable for medium or heavy castings. If a few castings of light section were desired and cast of this mixture, difficulty would be encountered due to chill. The addition of nickel in the ladle would remove this difficulty. It is well to bear in mind, nevertheless, that alloys are not cure-alls. They are, however, valuable tools for special applications and must be used with proper care.

CORROSION

MR. MOLDENKE.—I should like to ask Mr. Forrest if he has not other samples of pipe also, which show entirely different corrosion results. Thus, the famous Versailles pipe line of Louis XIV, which is practically as good to-day as it was in 1664 to 1686. In making examinations of existing pipe lines, I often have pieces cut out of them and find excellent conditions, unless damage has been done by stray currents, through electrolysis, and then such damage appears within a year or two after the laying of the pipe, or putting down the trolley tracks.

MR. H. O. FORREST.[1]—We have in our collection but two samples of foreign pipe, one French and the other Belgian.

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[1] Director, Research Laboratory of Applied Chemistry, Massachusetts Institute of Technology, Cambridge, Mass.

THE EFFECT OF REDUCTION FROM INGOT TO FORGING IN STEEL FORGINGS

By Lawford H. Fry.[1]

Synopsis

The paper summarizes an extensive series of investigations carried out by a sub-committee of the Society's Committee A-1 on Steel to ascertain how the properties of steel forgings are affected when a variation is made in the amount of reduction between ingot and forging.

The general problems involved are discussed and similar work done by other investigators is considered.

It is concluded that in specifications for steel forgings it is unnecessary and may be undesirable to specify the amount of reduction to be made between ingot and forging, but that in specifications for blooms a requirement of a three to one reduction from ingot to bloom may be justified.

Many of the Society's specifications have developed gradually from older specifications. In such cases there is a possibility of clauses remaining in the specifications because of the preservative effect of tradition and not because they correspond to essential requirements of present practice.

Some three years ago the attention of the Society's Committee A-1 on Steel was directed to Section 6 of the Standard Specifications for Carbon-Steel and Alloy-Steel Blooms, Billets and Slabs for Forgings (A 17 – 21).[2] This section requires that unless otherwise specified steel billets for forging purposes shall be produced from ingots having a cross-sectional area at least four times that of the billet. It was suggested that this requirement was an heritage from earlier days and that with modern methods it was no longer necessary or desirable. The question was referred to Sub-Committee VI on Steel Forgings and Billets. As there was found to be considerable difference of opinion in the sub-committee it was decided to carry out a series of experiments to throw light on the matter. A section under the chairmanship of R. P. Johnson was entrusted with the experimental work. This work was completed last year, and as a result Committee A-1 is now recommending to the Society that certain changes be made in Specifications A 17 – 21. It was impossible to go into details there as

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[1] Metallurgical Engineer, Standard Steel Works Co., Burnham, Pa.

[2] 1927 Book of A.S.T.M. Standards, Part I, p. 174.

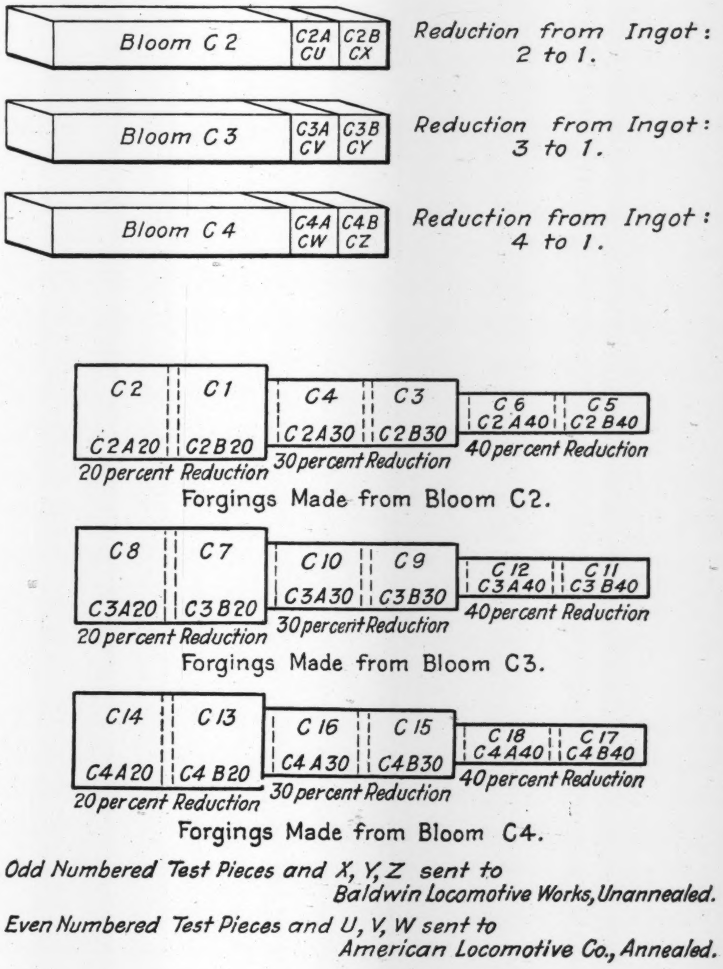
Bloom C 2
C2A CU
C2B CX
Reduction from Ingot: 2 to 1.
Bloom C 3
C3A CV
C3B CY
Reduction from Ingot: 3 to 1.
Bloom C 4
C4A CW
C4B CZ
Reduction from Ingot: 4 to 1.
C 2
C 1
C 2A 20
C2B 20
20 percent Reduction
C 4
C 3
C 2A30
C 2B30
30 percent Reduction
C 6
C 5
C 2 A 40
C 2 B40
40 percent Reduction
Forgings Made from Bloom C2.
C 8
C 7
C3A20
C 3 B 20
20 percent Reduction
C 10
C 9
C3A30
C3B30
30 percent Reduction
C 12
C 11
C 3 A 40
C 3 B 40
40 percent Reduction
Forgings Made from Bloom C3.
C 14
C 13
C4A 20
C4 B 20
20 percent Reduction
C 16
C 15
C4 A30
C4B30
30 percent Reduction
C 18
C 17
C 4 A 40
C 4 B 40
40 percent Reduction
Forgings Made from Bloom C4.
Odd Numbered Test Pieces and X, Y, Z sent to Baldwin Locomotive Works, Unannealed.
Even Numbered Test Pieces and U, V, W sent to American Locomotive Co., Annealed.

FIG. 1.—Diagram Showing Blooms and Forgings Provided for Test.

to the experimental work, but it was felt that this was of sufficient importance to be put on record, at least in part.

Four manufacturers of forgings—The Bethlehem Steel Co., Carnegie Steel Co., Standard Steel Works Co. and Camden Forge Co.—cooperated by furnishing the steel and doing the forging work required. In accordance with the program drawn up by the section each of the four companies was to furnish three ingots from one melt of its regular grade of forging steel, aiming at 0.50 per cent carbon and 0.60 per cent manganese. The three ingots from each company were to be of the same dimensions, but these dimensions were chosen individually by the companies to represent their regular practice in producing forgings. Reduction of the ingots to blooms was carried out by the companies in accordance with their regular practice. The cross-sectional area of the blooms from one ingot of each group was to be one-half that of the original ingot area, for another ingot, one-third of the ingot area and for the third ingot of each group one-fourth of the ingot area. Pieces 24 in. long were to be taken from the blooms thus produced and the remaining portions were to have their cross-sectional areas reduced by forging to a circular cross-section in three steps, the diameters being such as to give a reduction of cross-sectional area of 20 per cent of the bloom area in the first step, 30 per cent in the second step and 40 per cent in the third. The length of each step was to be twice the diameter.

When the forging work was completed tension test specimens were to be taken from the blooms and forgings in the unannealed condition and then blooms and forgings were to be annealed and tension test specimens taken.

The diagram in Fig. 1 shows the arrangement of blooms and forgings to be provided. The annealing was to be done by the American Locomotive Co., the tension test specimens were to be cut out by The Baldwin Locomotive Works, chemical analysis of each tension specimen was to be made by the New York Central Lines, and longitudinal and transverse micrographs from each specimen by the Pennsylvania Railroad Co. The original program called for a simple anneal, but as the micrographs showed that this treatment had not been entirely satisfactory in breaking up the grain, the unannealed portions of the blooms and forgings were normalized and tempered, and tension tests and micrographs were taken from them in this condition. The tension tests of the untreated and of the annealed specimens were made by the Tinius Olsen Testing Machine Co., while the tests of the normalized-and-drawn specimens were made by the American Locomotive Co.

The amount of work involved in the investigation was considerable. The original program called for 12 blooms, 36 forgings, 96 tension tests, 96 chemical analyses, and 192 micrographs. The normalizing and drawing added 48 tension tests and 96 micrographs.

After consideration of the results obtained, the section reported that the amount of reduction from ingot to forging had very little effect on the values obtained for the yield point and the tensile strength. The test results showed, however, that as the amount of reduction was increased there was a slight improvement in the ductility as measured by the values found for the elongation and reduction of area. This improvement of ductility increased with the low ratios of reduction between ingot and forging, but when the ratio of reduction was increased above three to one the gain in ductility was practically negligible, the increase, if any, being not greater than the difference in ductility values which would normally occur between the values obtained from different specimens taken from the same forging. It was pointed out that as each manufacturer had used one size of ingot for all forgings, the forgings which had the greatest reduction from the ingot had consequently the least cross-section. Part of the increase of ductility with increasing ratio of reduction might, therefore, be due to the smaller size of the forging which would permit a more effective heat treatment.

In conclusion the sub-committee expressed the opinion that in forging specifications which set definite requirements as to tensile properties there was no necessity for specifying a requirement as to the amount of reduction from ingot, but that in specifications for billets it was desirable to protect the purchaser by requiring a reduction of three to one from ingot to billet.

It will be seen that the sub-committee in arriving at its conclusions had before it a very considerable body of experimental results, and it is the purpose of the present paper to put on record more of these than could be covered in the report of Committee A-1.

The quality of the forgings produced was judged by the tension tests and by the micrographs. Selections of these are therefore included herewith.

Before presenting in detail the experimental results obtained by the section, the author feels it desirable to offer a general discussion of the principles involved and to consider some of the work done on the subject by other investigators. It is to be understood that the section as a whole is not responsible for the facts and opinions which follow, except where its test results are quoted.

One of the most important contributions to the subject is a paper by Georges Charpy on "The Influence of Hot Deformation on the Qualities of Steel."[1] Charpy, after presenting a considerable amount of experimental work and a discussion of the principles involved, concludes that the effect on the properties of steel produced by hot working is quite complex. It depends on the original condition of the ingot as cast and on all of the subsequent deformation to which the ingot is subjected. The chief characteristic impressed on the steel by hot working is a strongly marked heterogeneity. That is to say, the differences between the properties found in test bars taken in different positions and directions throughout the steel are increased by an increased amount of working. For example the tensile properties of transverse test specimens are greatly reduced by increased working. Charpy concludes that "the total effect is far from being in the nature of a general improvement" as appears to be contemplated by specifications which set a minimum requirement for reduction from ingot to forging. "It would appear on the contrary," he says, "that the result is more injurious than useful, and that consequently the specification of a maximum deformation would be more logical." This is particularly true for forgings subject to transverse stresses.

Turning to a general consideration of the question, it is evident that the condition of the original ingot is of preponderant influence on the properties produced in the forging. The ingot as cast is necessarily non-uniform. By means of the selective freezing of the constituents of the steel the resultant structure and also the composition of the steel will vary throughout the ingot. A transverse section will show a dense columnar crystalline structure on the outside and a looser irregularly crystalline structure in the center. In addition there will of necessity be a certain amount of non-metallic inclusions scattered throughout the ingot. Hot working, whether by rolling or by forging, will distort the ingot structure, elongating it in the direction in which the metal is most drawn out. Fiber is thus produced, but it seems improbable that any of the original characteristic heterogeneity of the ingot is removed or completely obliterated. Elements of weakness in the original ingot will still be present in the forging no matter what reduction is made, but they will be arranged differently. If the ingot is poorly made, with open structure and numerous slaggy inclusions, the forging will have a marked woody fiber. If sufficiently drawn out it may give fairly good tensile properties longitudinally, but these will be markedly inferior in the transverse direction. An ingot of dense clean steel will produce less fiber, will give satisfactory properties longitudinally with a small amount of working, and will give comparatively high tensile properties transversely. These statements indicate that reduction in area from the ingot by hot working may serve to reduce, in the longitudinal direction, the effect of defects in the ingot, but none of the imperfections will be eliminated and their effect in the transverse direction will be increased. In this connection it should be noted that if forgings of a given size are to be produced, an increase in the ratio of reduction requires an increase in the size of the ingot, and an increase in the size of the ingot increases the heterogeneity of structure and composition of the steel.

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[1] *Journal*, Iron and Steel Inst., Vol. XCVIII, p. 7 (1918).

TABLE I.—DETAILS OF MANUFACTURE OF EXPERIMENTAL BLOOMS AND FORGINGS.

	Manufacturer A		Manufacturer B		Manufacturer C		Manufacturer D	
	Basic Steel Ingot, 19 by $20\frac{1}{2}$ in., 390 sq. in., Rolled		Basic Steel Ingot, 22 by 22 in., 476 sq. in., Rolled		Basic Steel Ingot, 22 in. diameter, 380 sq. in., Pressed		Acid Steel Ingot, 23 in. diameter, 420 sq. in., Pressed	
	Size, in.	Cross-sectional Area, sq. in.	Size, in.	Cross-sectional Area, sq. in.	Size, in.	Cross-sectional Area, sq. in.	Size, in.	Cross-sectional Area, sq. in.
2 to 1 Blooms...........	14 by 14	196	$15\frac{3}{8}$ by $15\frac{3}{8}$	236	$13\frac{3}{4}$ by $13\frac{3}{4}$	189	$14\frac{1}{2}$ by $14\frac{1}{2}$	210
3 to 1 Blooms...........	$11\frac{1}{2}$ by $11\frac{1}{2}$	132	$12\frac{5}{8}$ by $12\frac{5}{8}$	159	$11\frac{1}{4}$ by $11\frac{1}{4}$	127	$11\frac{3}{4}$ by $11\frac{3}{4}$	138
4 to 1 Blooms...........	10 by 10	100	11 by 11	121	$9\frac{3}{4}$ by $9\frac{3}{4}$	95	$10\frac{1}{4}$ by $10\frac{1}{4}$	105
FORGINGS								
2 to 1 Blooms:								
20 per cent Reduction	14 diam.	154	$15\frac{3}{8}$ diam.	186	$13\frac{3}{4}$ diam.	149	$14\frac{1}{2}$ diam.	165
30 per cent Reduction	$13\frac{3}{16}$ diam.	136	$14\frac{1}{2}$ diam.	165	13 diam.	133	$13\frac{1}{2}$ diam.	143
40 per cent Reduction	$12\frac{1}{8}$ diam.	115	$13\frac{1}{2}$ diam.	143	$11\frac{3}{4}$ diam.	108	$12\frac{1}{2}$ diam.	122
3 to 1 Blooms:								
20 per cent Reduction	$11\frac{1}{2}$ diam.	104	$12\frac{5}{8}$ diam.	125	$11\frac{1}{4}$ diam.	99	$11\frac{3}{4}$ diam.	108
30 per cent Reduction	$10\frac{3}{4}$ diam.	91	$11\frac{7}{8}$ diam.	111	$10\frac{1}{2}$ diam.	87	11 diam.	95
40 per cent Reduction	10 diam.	79	11 diam.	95	$9\frac{3}{4}$ diam.	75	$10\frac{1}{4}$ diam.	83
4 to 1 Blooms:								
20 per cent Reduction	10 diam.	79	11 diam.	95	$9\frac{3}{4}$ diam.	75	$10\frac{1}{4}$ diam.	83
30 per cent Reduction	$9\frac{3}{8}$ diam.	69	$10\frac{3}{8}$ diam.	85	$9\frac{1}{4}$ diam.	67	$9\frac{3}{4}$ diam.	75
40 per cent Reduction	$8\frac{5}{8}$ diam.	58	$9\frac{5}{8}$ diam.	73	$8\frac{1}{2}$ diam.	57	9 diam.	64

Discussing the question, Henry Marion Howe[1] says: "Turning to intelligent practice, these benefits of casting in small ingots are thought to outweigh the accompanying decrease in the kneading or work which can be put on the metal before it reaches its final shape." The final quality of a forging is determined by the combination of two factors, first, the macrostructure which is that of the ingot structure deformed by forging, and second, the microstructure produced by heat treatment. Evidence from the present tests and from other sources indicates that the final microstructure is dependent primarily on the heat treatment and on the size of the forging, and is not greatly affected by the amount of mechanical work that the forging may have received. Charpy in the paper referred to above quotes Howe to the effect that the supposed special effect of kneading and pressure does not exist or is relatively unimportant.

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[1] H. M. Howe, "Metallography of Steel and Cast Iron," p. 554.

TABLE II.—TENSILE PROPERTIES OF NORMALIZED-AND-DRAWN FORGINGS. AVERAGE FOR ALL FOUR MANUFACTURERS.

REDUCTION, BLOOM TO FORGING		REDUCTION, INGOT TO BLOOM, 2 TO 1	REDUCTION, INGOT TO BLOOM, 3 TO 1	REDUCTION, INGOT TO BLOOM, 4 TO 1
Bloom........	Yield Point, lb. per sq. in.......	43 290	42 820	47 890
	Tensile Strength, lb. per sq. in..	79 320	84 820	87 640
	Elongation in 2 in., per cent....	23.0	24.5	22.8
	Reduction of Area, per cent.....	35.9	44.1	41.6
20 per cent...	Yield Point, lb. per sq. in.......	46 575	45 740	45 450
	Tensile Strength, lb. per sq. in..	80 950	85 700	83 950
	Elongation in 2 in., per cent....	25.0	25.3	24.5
	Reduction of Area, per cent.....	41.7	44.3	42.9
30 per cent...	Yield Point, lb. per sq. in.......	42 155	45 680	45 100
	Tensile Strength, lb. per sq. in..	80 540	87 670	85 820
	Elongation in 2 in., per cent....	27.0	26.0	23.5
	Reduction of Area, per cent.....	46.7	42.8	42.3
40 per cent...	Yield Point, lb. per sq. in.......	44 750	44 495	44 580
	Tensile Strength, lb. per sq. in..	83 060	86 610	88 270
	Elongation in 2 in., per cent....	25.0	24.8	24.1
	Reduction of Area, per cent.....	42.6	42.9	42.1

Unfortunately, owing to the conditions involved, it is not possible to make direct experimental measurement of the effect of forging reduction. The forging reduction cannot be varied without varying one other factor in the problem. If forgings of given dimensions are produced with various ratios of reduction from the ingot, ingots of different sizes and therefore of different original structures must be used. If ingots of the same size are given different reductions, forgings of different sizes are produced and the smaller sized forgings respond more readily to the heat treatment.

In the experiments carried out by the section each maker used one size of ingot and varied the reduction by varying the diameters of the forgings produced. The sizes of ingots, blooms and forgings are shown in Table I. The cross-sectional areas of the ingots ranged from 380 to 476 sq. in. with an average of 416 sq. in. for all four makers. On this basis the average cross-sectional area of blooms and forgings was as follows:

REDUCTION, BLOOM TO FORGING, PER CENT	CROSS-SECTIONAL AREA OF FORGING, SQ. IN.		
	Reduction, Ingot to Bloom, 2 to 1	Reduction, Ingot to Bloom, 3 to 1	Reduction, Ingot to Bloom, 4 to 1
....	208	139	104
20	166	111	83
30	145	97	72
40	125	83	63

The values of the tensile properties of the normalized-and-drawn forgings are shown in Table II, and to facilitate comparison the values of the so-called "quality factors" have been computed and are compiled below. These quality factors are (1) elongation multiplied by tensile strength, and (2) reduction of area multiplied by tensile strength:

REDUCTION, BLOOM TO FORGING		REDUCTION, INGOT TO BLOOM, 2 TO 1	REDUCTION, INGOT TO BLOOM, 3 TO 1	REDUCTION, INGOT TO BLOOM, 4 TO 1
Bloom	Elongation × Tensile Strength	1 820 000	2 080 000	2 000 000
	Reduction of Area × Tensile Strength	2 850 000	3 740 000	3 650 000
20 per cent	Elongation × Tensile Strength	2 020 000	2 160 000	2 060 000
	Reduction of Area × Tensile Strength	3 370 000	3 800 000	3 600 000
30 per cent	Elongation × Tensile Strength	2 170 000	2 280 000	2 240 000
	Reduction of Area × Tensile Strength	3 750 000	3 750 000	3 640 000
40 per cent	Elongation × Tensile Strength	2 080 000	2 150 000	2 130 000
	Reduction of Area × Tensile Strength	3 530 000	3 530 000	3 720 000

These values of the quality factors are plotted in Fig. 2. The abscissas are given two sets of values: on the lower scale the actual cross-sectional area of the forgings, and on the upper scale the ratio of reduction from the ingot to the forging. The values from the bloom with a 2 to 1 reduction and from the forging with 2 to 1 bloom reduction and 20 per cent forging reduction are low. In these two cases the reduction from ingot to forging is less than 2.5 to 1. For all the other blooms and forgings with reductions greater than this the amount of reduction has little influence on the values of the quality factors. For the points corresponding to reduction of area multiplied by tensile strength the best mean line will differ but little from a horizontal line with the value 3,690,000. For the elongation multiplied by tensile strength the best mean line will run from a value of approximately 2,120,000 at a reduction of 3 to 1 up to 2,200,000 at a reduction of 7 to 1. The variation is slight and may very well be due to the better heat treatment due to the smaller cross-section of the forgings with the greater reductions. As confirmatory of this view, results of an experiment made recently at the Standard Steel Works are given. In this case four forgings all having the same diameter, 5 in., were made from ingots from the same melt ranging in diameter from 10 to 23 in. The respective values of the quality factors from the normalized-and-drawn forgings are given below. The values are averages from two tests from each forging:

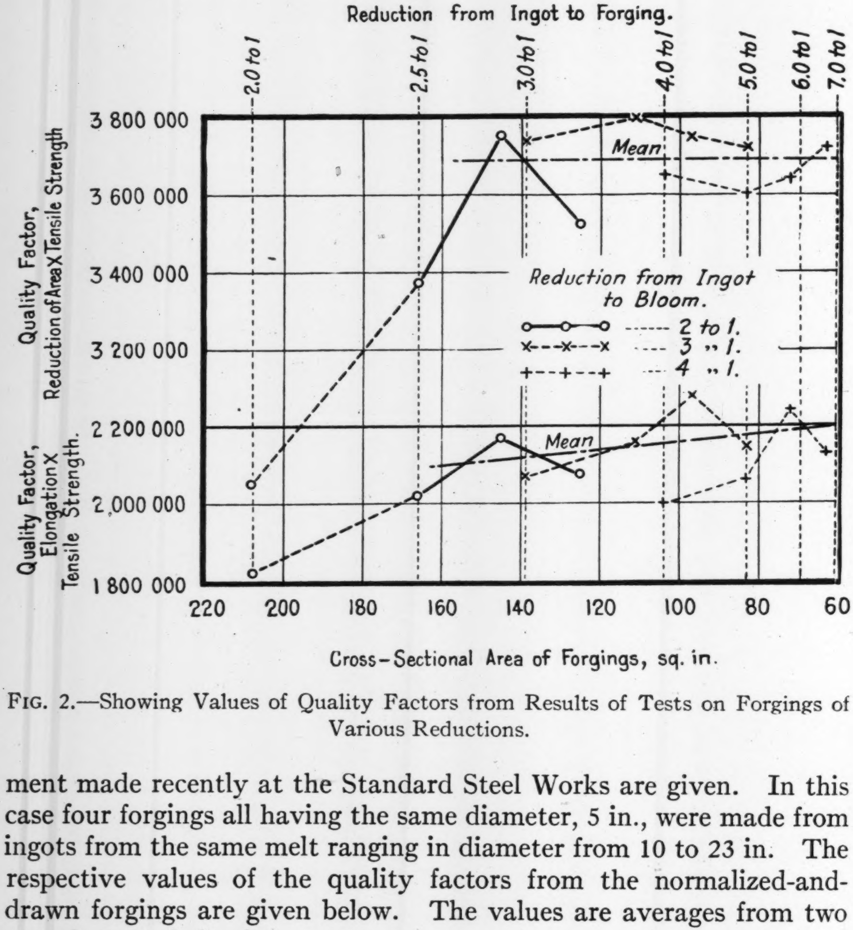
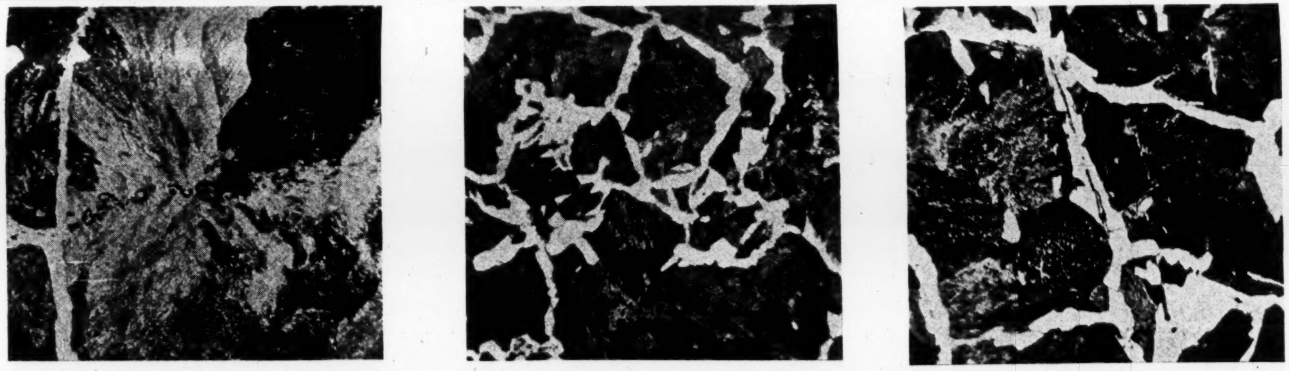


FIG. 2.—Showing Values of Quality Factors from Results of Tests on Forgings of Various Reductions.

INGOT DIAMETER, IN.	FORGING REDUCTION	TENSILE-STRENGTH MULTIPLIED BY ELONGATION	TENSILE STRENGTH MULTIPLIED BY REDUCTION OF AREA
10	4 to 1	2 490 000	4 640 000
14	7 to 1	2 420 000	4 600 000
20	16 to 1	2 420 000	4 600 000
23	20 to 1	2 360 000	4 020 000

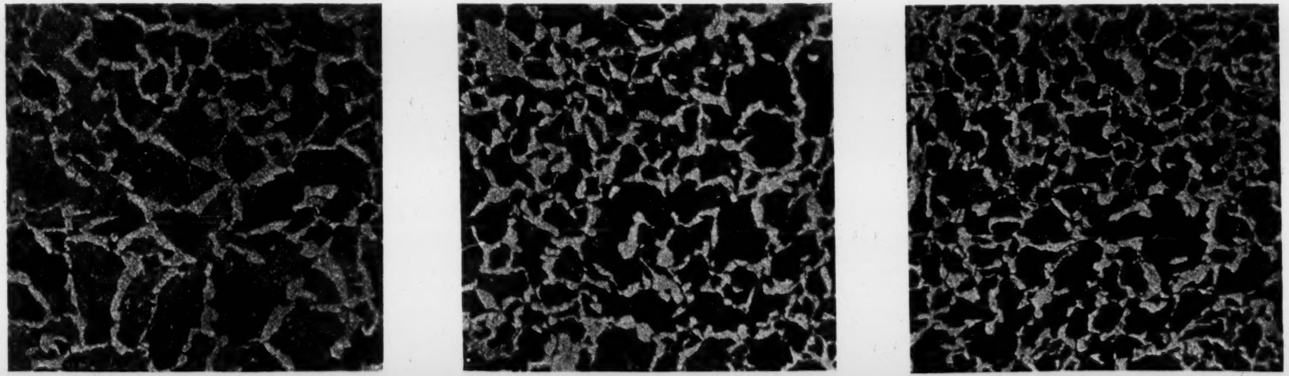


(*a*) Reduction, ingot to bloom, 2 to 1; bloom to forging, none; total reduction, 2 to 1.

(*b*) Reduction, ingot to bloom, 3 to 1; bloom to forging, 20 per cent; total reduction, 3.8 to 1.

(*c*) Reduction, ingot to bloom, 4 to 1; bloom to forging, 30 per cent; total reduction, 5.6 to 1.

Unannealed



(*d*) Reduction, ingot to bloom, 2 to 1; bloom to forging, none; total reduction, 2 to 1.

(*e*) Reduction, ingot to bloom, 3 to 1; bloom to forging, 20 per cent; total reduction, 3.8 to 1.

(*f*) Reduction, ingot to bloom, 4 to 1; bloom to forging, 30 per cent; total reduction, 5.6 to 1.

Normalized and Drawn

FIG. 3.—Photomicrographs of Forgings at Various Ratios of Reduction (× 100).

Here the smallest reduction gives the best results which are undoubtedly due to the advantage of the smaller ingot.

In conclusion the micrographs of Fig. 3 are presented, but with certain reservations. The full set of micrographs obtained by the section shows that the heat treatment of the forgings was not entirely satisfactory. In several cases, although the tensile properties were acceptable, the micrographs indicated that the grain refinement had not been carried as far as was desirable. The micrographs reproduced are all from the product of one manufacturer and have been chosen as representative of forgings in the unannealed condition and when normalized and drawn. The pieces represented have the following characteristics:

		Reduction, Ingot to Bloom	Reduction, Bloom to Forging, per cent	Total Reduction from Ingot	Cross-sectional Area, sq. in.
Figs. 3 (*a*) and (*d*)	Bloom	2 to 1	—	2.0 to 1	236
Figs. 3 (*b*) and (*e*)	Forging	3 to 1	20	3.8 to 1	125
Figs. 3 (*c*) and (*f*)	Forging	4 to 1	30	5.6 to 1	85

In the unannealed condition the two forgings show little difference in microstructure, while the bloom has very considerably larger grains. This is probably due to the fact that the bloom being rolled with a reduction of only 2 to 1 was finished at a higher temperature. Similarly in the annealed condition, the two forgings, despite a considerable difference in total reduction, show very similar microstructure. The bloom has a larger grain. The values of the tensile properties of this bloom were low. Results from other blooms indicate, however, that with more effective annealing a better microstructure could have been obtained, and the tensile properties could have been brought up to meet the requirements of the forging specifications.

It seems reasonable to conclude with Howe that the kneading action has little specific effect. The microstructure is mainly dependent on the heat treatment, but it must be recognized that if the final mechanical work is done at a proper temperature the piece will be more susceptible to effective heat treatment than if it is finished at an abnormally high temperature.

From consideration of the foregoing it appears that a manufacturer of forgings will do well to adapt the size of ingot used to the forgings to be produced, and that the purchaser of forgings is protected by the specification requirements as to physical properties. The purchaser of billets to be worked up into forgings is in a slightly different position, and therefore a general specification for billets may well contain the requirement that the cross-sectional area of the ingot shall be not less than three times that of the billet produced.

DISCUSSION

MR. F. N. SPELLER.[1]—I should like to ask how the transverse strength varies with the ingot reduction from ingot to bloom.

MR. L. H. FRY.[2]—In the sub-committee work, no transverse tests were made, but there is plenty of other evidence to show that the values of the transverse properties are reduced by increasing the amount of reduction.

If tests are taken from an ingot before any forging work is done the tensile properties will be the same whether the test specimen is taken parallel to the long axis of the ingot or transversely to this direction. Then if the ingot is worked down and longitudinal and transverse tests are taken at various stages it will be found that the values of the tensile strength and of the yield point will be very little affected by the reduction and will be practically the same for both the longitudinal and transverse directions. The ductility, as measured by the elongation and reduction of area will vary with the forging reduction. In the longitudinal direction the ductility will increase slightly with the amount of reduction at first, but will show little increase when the reduction is carried beyond a ratio of about three to one. In the transverse direction the ductility falls off as the reduction increases and may reach very low values for high ratios of reduction.

MR. O. C. CROMWELL.[3]—What consideration was there in connection with the rate of heating of the billets? Was the rate of heating controlled? Did the billets have time to properly absorb the heat? Was any attempt made to measure that one element? My observation is that this is a very important part of the work that you are doing, to see that the heat is properly introduced into the forgings, that is, that you have a well-soaked forging. I would ask another question, what is the result of the fatigue tests? Were any fatigue tests made?

MR. FRY.—Heating of the billets and forgings was in accordance with the regular practice of the manufacturers. No fatigue tests were made. Fatigue testing is still in the laboratory stage and we are not yet in a position to correlate fatigue results and serviceability. The tension tests made to compare the forgings are such as are currently used to compare and to govern the acceptance of forgings to be used.

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[1] Director, Department of Metallurgy and Research, National Tube Co., Pittsburgh, Pa.

[2] Metallurgical Engineer, Standard Steel Works Co., Burnham, Pa.

[3] Assistant to Chief of Motive Power and Equipment, Baltimore and Ohio Railroad, Baltimore, Md.

MR. CROMWELL.—Fatigue is what we get in service, this test should be extended into a service test or something that is equivalent. We may decide upon a certain grade or analysis of steel to do a certain work, but we do not know anything about it until we put it into service and find out, and it ought to be subject to service condition. It would be well if the investigation were extended a little further, to look into that phase.

(*By letter*).—The limited number of tests presented together with the very few micrographs exhibited give the members of the Society very little information on this most important subject of reduction from ingot to bloom.

The physical test as represented in the tension testing machine and the study of the micrographs do not appear to me to give the value that is needed; that is, something approximated in service conditions the metal is subjected to, such as would be developed in a fatigue or vibratory test.

It is singular that the driving axles that we have nowadays fail in very early service, very much earlier than under the older locomotives, and it would seem probable that these failures may be due to what may be designated as skin stresses wherein the metal may be highly stressed on the surface, resulting in the breaking down of the structure which tension tests will not disclose, and what we need is a test that would develop the strength of the metal to stand this service.

TENSION, BEND AND IMPACT TESTS ON REINFORCEMENT BARS[1]

By Willis A. Slater[2] and George A. Smith[3]

Synopsis

A standard test for quality of reinforcement bars is that of bending the bars, without heating, through 90 or 180 deg. with a radius of bend equal to 1, 2, or 3 diameters of the bar. The angle and radius of the bend depends generally upon the size and hardness of the bar. The difference in the angle and radius of the bend required for different types of bars causes differences in the severity of the tests applied to the different bars. Thus it seems that the tendency is to suit the specifications to what the bars of different grade will meet successfully rather than to the service demanded of the bars in bending for use as reinforcement.

In an effort to find a method of making the cold bend test of uniform severity for bars of all sizes and degrees of hardness, and the finding of an impact test suitable for detecting brittleness in bars, a series of tests was made at the U. S. Bureau of Standards on bars purchased in the open market. In these tests it was desired to find out also whether the application of the cold bend test might impair the tensile strength even though the bar did not fail in the bend test.

Cold bend, tension, and impact tests were made. The cold bend tests were made by the standard methods and by means of an improved machine designed and built at the Bureau of Standards.

Bars which did not break in the cold bend test did not have their strength seriously affected by the bending. With the special bending machine used, all bars, regardless of their size, appeared to undergo about the same strain when bent through the same angle.

The apparent yield point in the bending test was much higher than the yield point determined in the direct tension test. The impact test used gave an approximate measure of the yield point of the bar, but none of the bars broke in the impact test except a few which had been quenched in water from a high temperature.

Introduction

The main function of a cold bend test for reinforcement bars should be to ascertain whether bars will meet safely the conditions imposed upon them when they are bent commercially for use as reinforcement. The present trend in specifications for design of reinforced concrete is toward the bending of reinforcement bars to a radius which is stated as not less than a certain number of times the diameter of the bar. It seems reasonable, therefore, that in specifying a bend test designed to give this information the ratio of the diameter of the bend to the diameter of the bar should be the same for bars of all sizes and all yield points. It seems logical also that the test should be specified in such a manner that in bars of all sizes the bending through a certain angle would subject them all to strains of the same magnitude. If the standard tests for all bars have thus been reduced to a uniform basis, the suitability of a bar for a given purpose may be judged by its behavior in such a test.

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[1] Publication Approved by the Director of the Bureau of Standards of the U. S. Department of Commerce.

[2] Director of Fritz Engineering Laboratory, Lehigh University, formerly Engineer Physicist, U. S. Bureau of Standards, Washington, D. C.

[3] Hydraulic Cement Engineer, Johns-Manville Corp., Manville, N. J., formerly Associate Engineer Physicist, U. S. Bureau of Standards, Washington, D. C.

The present specifications for reinforcment bars do not generally furnish this uniform criterion. Instead the bars of small size and low yield point are required to be bent to a smaller radius and through a larger angle than are those of large size and high yield point, thus to some extent suiting the test to the properties of the bar rather than to the service to be performed.

The tests reported in this paper were made at the Bureau of Standards, Washington, D. C., in 1921 and 1922 with the purpose of finding a cold bend test which at the same time that it requires all bars to be bent to the same radius would also subject them to strains of the same magnitude regardless of the sizes or yield points of the bars. In these tests it was desired to find out also whether the application of the cold bend test might impair the tensile strength of the bar although the bar did not fail in the bend test.

Attention was given also to finding an impact test which would detect brittleness in bars, and which would subject all bars to strains of the same magnitude regardless of the size of the bar.

SCOPE OF TESTS

Tension, bend and impact tests were made on reinforcement bars ranging from $\frac{3}{8}$ to 1 in. in diameter, and in yield point from about 36,000 to 96,000 lb. per sq. in. There were nine lots (lots A, B, C, D, E, F, G, H, and J) of nominally round bars $\frac{3}{8}$, $\frac{1}{2}$, $\frac{3}{4}$ and 1 in. in diameter, and one lot of $\frac{7}{8}$-in. square bars. The bars were purchased in the open market from ten different sources. It had been planned to use plain bars throughout, but owing to the difficulty of securing plain bars some deformed bars were included in the tests.

Six kinds of tests requiring four specimens were made on each bar:

Test No. 1.—Specimens 43$\frac{1}{2}$ in. long were bent cold about a pin three times the diameter of the specimens in an Olsen cold bend testing machine.

Test No. 2.—Cold bend tests on bars $40\frac{1}{2}$ in. long similar to Test No. 1 were made, except that the bars were bent in a special bending machine designed at the Bureau of Standards.

Test No. 3.—Impact tests were made on 18-in. lengths for the $\frac{3}{8}$ and $\frac{1}{2}$-in. bars and on 36-in. lengths for the $\frac{3}{4}$ and 1-in. bars.

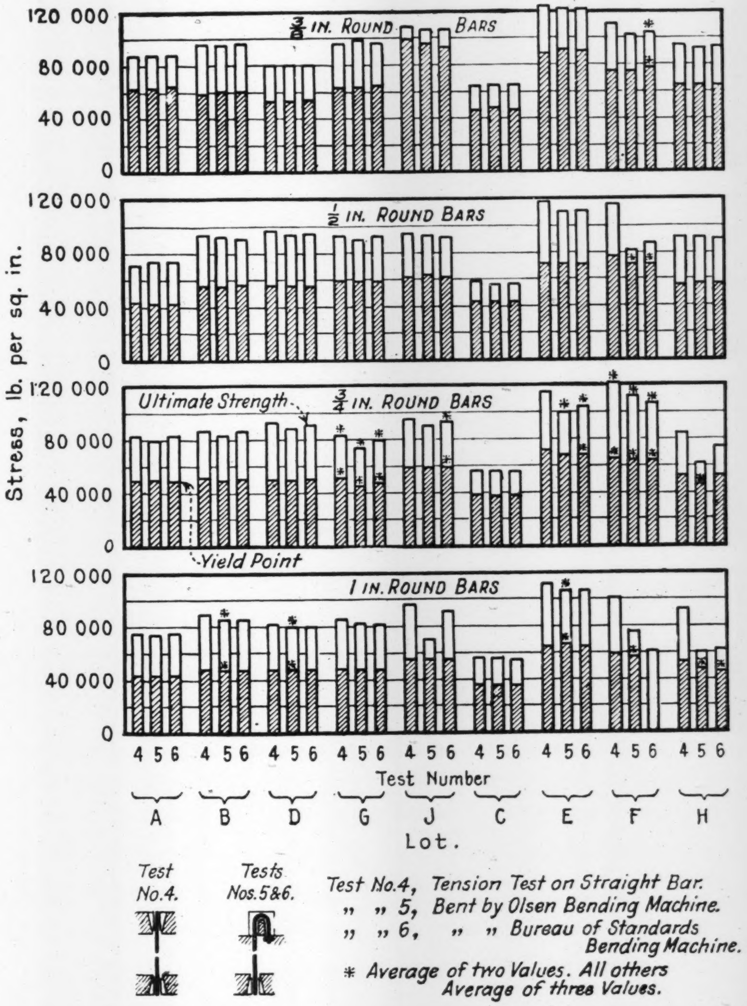


FIG. 1.—Showing Yield Points and Ultimate Strengths Obtained from Tension Tests.

Test No. 4.—Tension tests were made on straight specimens 24 in. long.

Test No. 5.—Tension tests were made on the specimens previously bent in the Olsen bending machine described under Test No. 1.

Test No. 6.—Tension tests were made on specimens previously bent in Test No. 2 with the special bending machine designed at the Bureau of Standards.

In marking the bars for identification each lot of bars was given two letters, the first to identify the lot and the second, either A, B or C, to designate the three bars of the same lot and of the same size. The bars were then cut into specimens having lengths of $43\frac{1}{2}$, $40\frac{1}{2}$, 36 and 24 in. for tests Nos. 1 to 4, in the order given. Each $\frac{1}{2}$-in. and each $\frac{3}{8}$-in. impact specimen was then cut in two again to give two specimens 18 in. long. For the 1-in. and $\frac{3}{4}$-in. bars only one impact specimen was prepared, since the length of 36 in. was not sufficient to furnish two specimens each for testing on a span of 24 diameters.

TABLE I.—TEST RESULTS FOR $\frac{7}{8}$-IN. SQUARE BARS.

Bar	Tension Test No. 4		Bend Test No. 2		Permanent Deflection under Impact, in.
	Yield Point, lb. per sq. in.	Tensile Strength, lb. per sq. in.	Angle of Bend, deg.	Yield Point, lb. per sq. in.	
No. 1	62 290	110 850	180[a]	129 000	0.271
No. 2	63 690	125 170	91	129 000	0.291
No. 3	62 790	111 020	180[a]	129 000	0.264
No. 4	63 410	120 000	104	137 000	0.196
No. 5	65 330	126 720	120	129 000	0.185
No. 6	63 730	120 550	81	129 000	0.209
No. 7	63 830	121 180	165[a]	137 000	0.212
No. 8	65 110	121 630	69	137 000	0.191
No. 9	74 930	129 960	114	145 000	0.164
No. 10	63 830	121 830	104	129 000	0.184
No. 11	63 100	121 470	130	137 000	0.186
No. 12	64 670	116 940	83	129 000	0.171
No. 13	80 980	139 100	7	161 000	0.126
No. 14	62 180	120 160	130	121 000	0.197
No. 21	65 620	126 260	35	145 000	0.167
No. 30	80 820	139 210	65	153 000	0.128
No. 32	77 150	130 610	69	153 000	0.163
No. 34	64 400	125 390	100	137 000	0.179
No. 35	63 390	119 860	74	121 000	0.216
No. 36	62 930	119 750	103	121 000	0.212
No. 39	66 930	124 730	38	145 000	0.174
No. 40	64 520	121 480	69	129 000	0.204
Average	66 600	123 300		135 700	

[a] Bars unbroken.

From these lots three tests of each kind were made, except as noted in Fig. 1. With the $\frac{7}{8}$-in. square bars only the tension test on the straight specimens, the cold bend test in the special bending machine and the impact tests were made. Generally it was not possible to make the tension test on the bent specimen because the bars broke before the 180-deg. bend was completed.

TENSION TESTS OF STRAIGHT SPECIMENS

The tension tests No. 4 were made in a 200,000-lb. Riehlé testing machine in which the $\frac{3}{8}$-in. and $\frac{1}{2}$-in. round specimens were caught between flat grips and the $\frac{3}{4}$-in. and 1-in. round specimens between "V" grips. In all tension tests only two speeds of the testing machine were used, "low" and "low-intermediate." The rate at which the movable head of the testing machine traveled under no load was 0.064 in. per minute for the low speed and 0.416 in. per minute for the low-intermediate speed when running idle. Load was applied at the low speed until the yield point of the specimen was reached, after which the test was continued at the low-intermediate speed until failure of the specimen occurred.

The yield points and tensile strengths for all the bars except the $\frac{7}{8}$-in. square bars are shown in Fig. 1. The data of the $\frac{7}{8}$-in. bars are given in Table I. The height of the shaded portion of each rectangle in Fig. 1 indicates the yield point and the extreme height the tensile strength. For cases in which no portion of the rectangle is shaded a yield point was not detected for one or more of the bars by the method used. The values plotted in Fig. 1 are the averages of three tests each, except in the cases marked with asterisks which are the average of two tests each. The above statement applies also to results to tension tests Nos. 5 and 6 on bent bars.

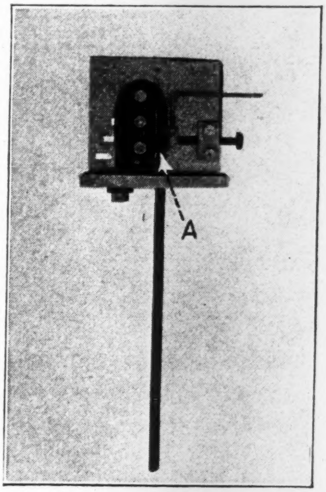


FIG. 2.—Block Used to Hold Bent Specimens for Tension Tests.

TENSION TESTS OF BENT SPECIMENS

The tension tests No. 5 were made on the specimens previously bent in an Olsen cold bend machine. The tests using test No. 6 were similar to those using test No. 5, except that the specimens were those previously bent in the special bending machine. Each specimen was threaded before bending in order to effect a positive anchor for the short leg of the hook in the tension test. The specimens were held in a special block that rested on the upper head of the testing machine. The arrangement is shown in Fig. 2. For all specimens the width of the block included within the bend of the bar was three times the diameter of the bar. The bent specimens were fastened in the block at the end of the short leg of the hook by means of a nut turned up tightly against the bearing plate. The long leg was left free to be gripped in the movable head of the testing machine. The load was then applied as in tests No. 4. In these tests the condition of testing was intended to simulate as nearly as possible that which exists when a hooked or bent bar is embedded in concrete and is subjected to a tensile stress. This is not strictly possible, since a hooked reinforcement bar is anchored by the bond between the steel and concrete along the bar as well as by the hook or bend. In these tests the holding of the short leg by means of the nut left the remainder of the specimen free except for friction, and large stresses were carried by the specimens at the threaded portion. This was shown by the failure of some of the specimens at the threaded portion or by the shearing of the threads in the nuts.

In a few cases the yield points were not detected because the specimens broke in the bends at loads below the yield point obtained in the straight tension tests, or because the bar slipped in the block, throwing the weighing beam out of balance at about the yield point load. In the others the yield points were within a few pounds of those obtained on the straight specimens. This is as should be expected, for it is not supposed that bending the bar would affect the yield point in direct tension. It is, therefore, in the tensile strength of the specimens that the effect of bending will be expected. In Fig. 3 are shown the ratios of the tensile strengths of the bent bars to the tensile strengths of the straight specimens. In most cases where the strength fell short of that of the straight specimens it can be attributed to one (in one instance to two) of the specimens breaking in the bend at from 19,000 to 45,000 lb. per sq. in. The remainder of the specimens in any particular lot broke at a load within 1 or 2 per cent of that shown for the straight specimens. In test No. 5 the 1-in. bars in lot J all broke at low loads but within 1500 lb. per sq. in. of the same tensile strength. The ratios, in most cases, do not show that the method of bending made a great deal of difference in the strengths of the bent specimens, but specimens bent in the special bending machine gave somewhat higher ratios than those bent in tests No. 1.

During the tests on the bent bars it was observed that a slipping took place over the block inside the bend. This slipping was measured on some of the specimens up to the yield point at approximately the point *A* in Fig. 2, and was as high as 0.21 in. for the $\frac{3}{8}$-in. bars and 0.86 in. for the 1-in. bars, but was usually about 0.15 in. and 0.40 in., respectively. In the bending tests the bars did not conform exactly to the mandrel about which they were bent and when set up for the tension test they did not exactly fit the interior block of the tension apparatus, as may be seen in Fig. 2. Upon the application of the load the bars were forced to conform approximately to the shape of

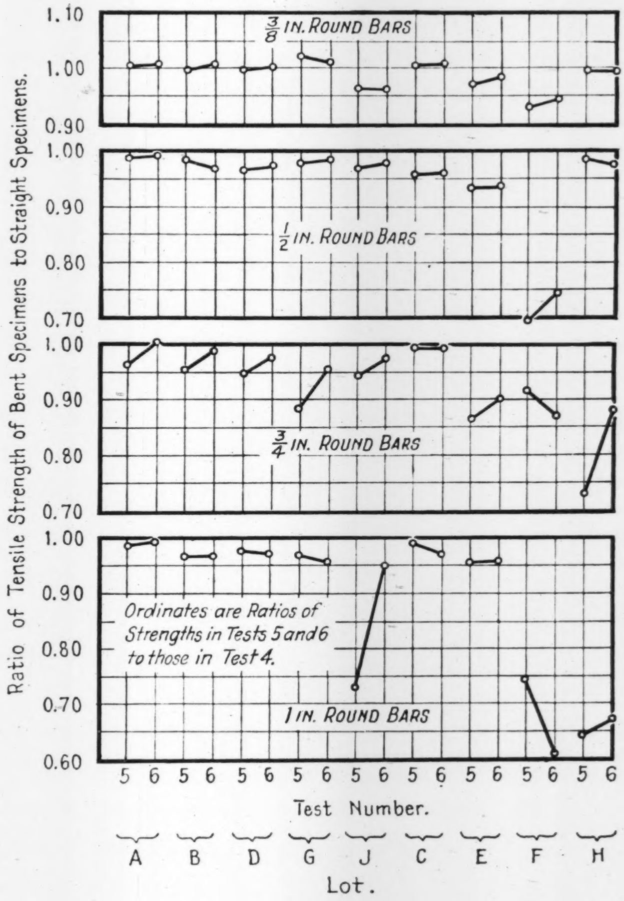


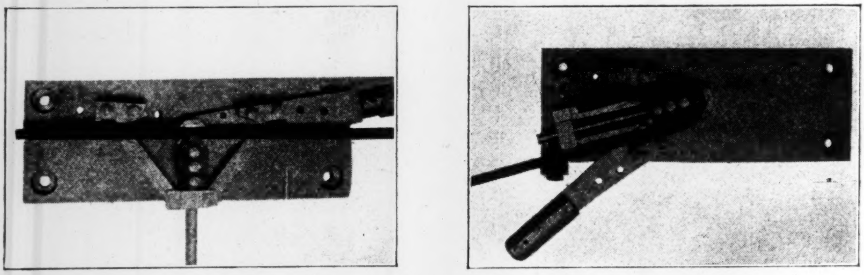
FIG. 3.—Showing Ratios of Tensile Strength of Bent Specimens to the Tensile Strength of Straight Specimens.

the block. The movement under load, together with the elongation under stress, caused the bars to slip around the block as pointed out above.

BENDING TESTS

The cold bend tests No. 1 were made in an Olsen cold bend testing machine. The pins around which the bars were bent were three times the diameter of the bar to be tested. When the specimen was placed in position and the movable pin which applied the bending force was brought into contact with it the moment arm (the distance from point of application of bending force to point of contact with mandrel) for each of the various bars was as follows:

⅜-in. round bar	3.28 diameters
½-in. round bar	2.86 diameters
¾-in. round bar	2.44 diameters
1-in. round bar	2.23 diameters



(*a*) Bar Ready for Bending. (*b*) After Bending.

FIG. 4.—Special Bending Machine.

On account of the movement of the point of application of the bending force relative to the bar a friction component was applied axially with the bar. This friction component increased as the diameters of the bars increased. Under these conditions the power was not sufficient to bend the 1-in. bars and it was necessary to modify the machine slightly. This modification appeared to be effective mainly in that it reduced the friction component.

All of the specimens, except one of the ¾-in. bars of group E, were bent through 180 deg. around a mandrel three times the diameter of the bar. That bar broke after bending through about 60 deg. It was observed that the bars without exception first scaled next to the mandrel and it was not until much later in any test that the tension side of the bar scaled. The scaling, too, extended a considerable distance from the mandrel.

The fact that in the Olsen bending machine the yield point stress (indicated by scaling) appeared earlier on the compression than on the tension side of the specimen gives some basis for expecting that the bending of bars for use as reinforcement might cause an injury to the metal which, while not clearly apparent, might seriously weaken the bar. An examination of the bars after bending disclosed no such injury, and the fact that in all cases the bars either broke completely in the bend test or else carried nearly as great a load after being bent as did the unbent bars may be taken as an indication that no serious danger would be present in a structure due to damage of the bars in bending.

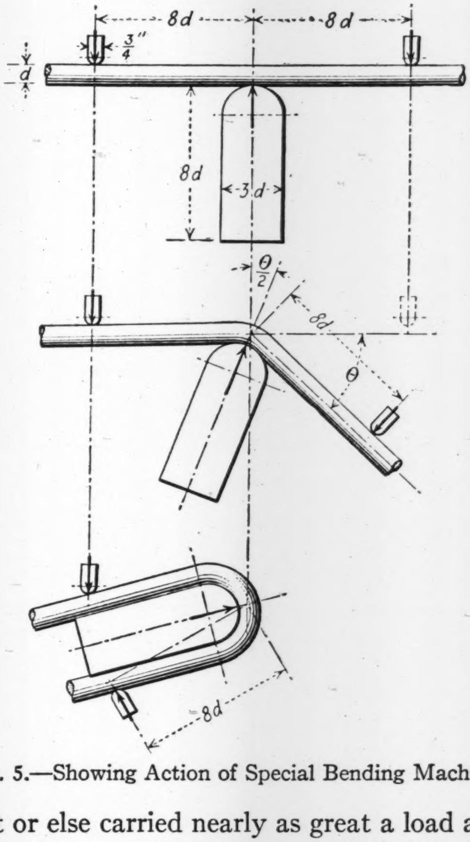


FIG. 5.—Showing Action of Special Bending Machine.

The cold bend tests No. 2 were made in a machine designed to produce equal stresses in bars of all sizes when bent through the same angle. It is believed that cold bend tests as usually made do not insure this condition. The bending device is shown in Fig. 4 and the principle of its operation in Fig. 5. At the beginning of the test the action is similar to that of a beam supported at both ends and loaded at the center of the span. As the lever is moved through an angle of θ deg. the mandrel moves through an angle of $\frac{\theta}{2}$ deg. The purpose of this arrangement was to secure the same conditions as to friction and slip at both points of support. It was observed while making the tests that the slip was small and approximately equally distributed. It is believed that the friction component was reduced to a negligible quantity. The diameter of the curve of the mandrels was maintained at three times the diameter of the specimen, as in test No. 1. The distance from the point of tangency between the bar and the mandrel to the point of loading was eight times the diameter of the test specimen.

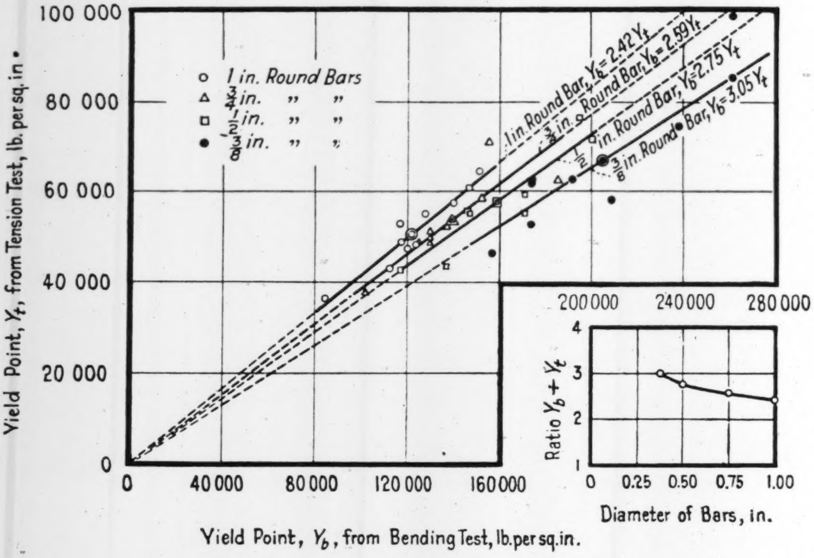


FIG. 6.—Showing Relation of Yield Points from the Bend Tests to the Yield Points from the Tension Tests.

In testing the bars with this machine it was observed that scaling occurred on the tension side and the compression side of the specimen almost simultaneously and progressed along the bar as the bending continued.

The bending of the bars in this test was done by hand and the pull required to start the bend was read on a spring balance 90 in. from the center of rotation of the loading lever. The outer fiber stress, when computed on the basis of beam action before the bar had bent through more than two or three degrees, is given by the equation:

$$f = \frac{917P}{d^3}$$

where P is the pull in pounds registered by the spring balance and d the diameter of the specimen in inches. This stress was computed for the load at which the bar appeared to yield and the relation of this stress to the yield point stress determined from the tension tests is shown in Fig. 6.

The pull required to stress the smaller rods up to the yield point was in some cases as small as 10 lb. Since the load could not be read closer than to 5 lb. the possible error of any observation on the $\frac{3}{8}$-in. bars may be as great as 50 per cent. With the larger bars, however, this proportional error would be much less and it will be seen that for them the points lie closer to a straight line than for the smaller bars. The determination of the amount of pull required was not made an essential part of the test and its value was not fully realized at the time of making the tests; therefore, it was not determined as accurately as it otherwise might have been.

If the condition aimed at were realized, that is, to produce the same stresses in bars of all sizes when bent through the same angle, then for the pull which caused the bar to yield, the ratio of the stress computed by the above equation to the yield point stress should be the same for all bars. That is, all the points in Fig. 6 should lie on a single line through the origin. It will be seen that the points may be grouped according to the sizes of the bars and that for any size of bar the points lie as close to a straight line as may be expected in view of the crudeness of the method of measuring the pull. The divergence of the lines for bars of different sizes also is not great and the indication is that the condition aimed at in the design of the machine was reasonably well, though not fully, realized.

Exclusive of the $\frac{7}{8}$-in. square bars, all the specimens except the one cut from the $\frac{3}{4}$-in. bar EA (lot E bar A) and the one from the $\frac{3}{8}$-in. bar FC successfully passed the 180-deg. bend test. The former broke at 120 deg. and the latter at 110 deg. Of the $\frac{7}{8}$-in. bars ten broke before bending through 90 deg.; nine broke after bending through 90 deg., and three were not broken after bending through 165 to 180 deg. Of the thirteen bars that had yield points below 65,000 lb. per sq. in., ten bent through 90 deg. or more and four broke between 69 and 83 deg.

After bending the $\frac{7}{8}$-in. square bars around the 3-diameter mandrel ($2\frac{5}{8}$ in.), the longer of the broken pieces of specimens Nos. 10, 13, and

21 were bent around a mandrel 3 in. in diameter, or 3.43 times the diameter of the bars. These bars broke after bending through 90, 47 and 116 deg., respectively, as compared with 104, 7 and 35 deg. about the smaller mandrel.

In Fig. 7 for each diameter of bar, the solid circle shows the highest yield point found for a specimen which broke in the bend test and the open circle shows the highest yield point for one which did not break in bending. For the $\frac{1}{2}$-in. and 1-in. bars there are no solid circles because no specimens of those sizes broke in bending. Only one $\frac{3}{8}$-in. and one $\frac{3}{4}$-in. bar broke in the bend test. Most of the $\frac{7}{8}$-in. bars broke before bending through 180 deg. and the highest, lowest and average of these are used in the diagram. This figure shows that of the three cases where bars broke in bending, in two cases it was the bars having the highest yield which broke. The limiting yield point was around 80,000 lb. per sq. in. Only the $\frac{3}{8}$-in. bars exceeded this yield point without breaking in bending. The average yield point for the $\frac{7}{8}$-in. bars which broke in bending is 67,200 lb. per sq. in. and none of the $\frac{7}{8}$-in. bars that did not break in bending had a yield point as great as this average. However, all three of the $\frac{7}{8}$-in. bars which did not break in bending had yield points higher than the lowest of those which broke.

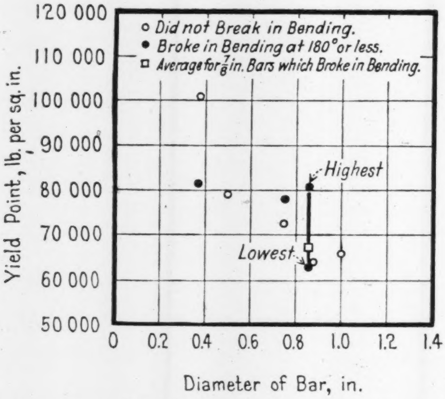


FIG. 7.—Yield Points for Bars Which Broke in Bending and for Bars Which Did Not Break in Bending.

Open circles show the highest yield points of bars which did not break in bending. Solid circles show the lowest yield points of bars which broke in bending except that for $\frac{7}{8}$-in. bars highest, lowest and average are shown.

The Society's specifications[1] require that bars larger than $\frac{3}{4}$ in.

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[1] Standard Specifications for Billet-Steel Concrete Reinforcement Bars (A 15 – 14), 1927 Book of A.S.T.M. Standards, Part I, p. 132.

need be bent only through 90 deg. around pins 3 and 4 times the diameters of the plain and deformed bars respectively. It is likely that in commercial bending the bars frequently receive more severe treatment than they do in the testing. It is possible, therefore, that bars which meet these specifications might break in commercial bending. For this reason, then, it seems desirable to give some consideration to the yield point as a criterion for judging whether a bar will be likely to break when bent in a commercial bending machine.

RELATION OF YIELD POINT IN BENDING TO THAT IN DIRECT TENSION

The yield point as determined by the bend tests was from about 2.4 to about 3.0 times as great as that determined by the direct tension tests, depending upon the size of the bar. The curve in the lower right-hand portion of Fig. 6 shows that the average ratio of the yield point from the bend tests to that in direct tension increased somewhat as the diameter of the bar decreased. While an exact agreement between the yield points as determined by bending and direct tension tests respectively is not to be expected, the difference noted was so great that a few independent tests were made in which the yield point in bending and that in direct tension could be determined more accurately, in order to see if the yield point in bending was as much greater than that in direct tension as the cold bent tests indicated that it was. Specimens were cut from a bar 1.01 in. in diameter and numbered consecutively according to their position from the end of the bar. Specimens Nos. 1 and 3 were tested in direct tension and specimens Nos. 2, 4, 6, and 8 in bending.

TABLE II.—COMPARISON OF YIELD POINT IN DIRECT TENSION WITH THAT IN BENDING.

Specimen	Kind of Test	Total Load at Yield Point, lb.	Stress at Yield Point, lb. per sq. in.
No. 1	Direct tension	37 760	47 100
No. 3	Direct tension	37 390	46 700
Average		37 575	46 900
No. 2	Bending	Not determined	
No. 4	Bending	1 342	79 800
No. 6	Bending	1 387	82 500
No. 8	Bending	1 395	83 000
Average		1 375	81 800

The tension specimens were 18 in. long. For the bend tests, the span was 24 in. and the load was applied at the center of the span in a testing machine with the head running at a speed (when idle) of about 0.024 in. per minute or less. In each case the yield point load was determined by "drop of weighing beam." The loads and computed yield point stresses are given in Table II. The agreement between tests of the same kind was good and the ratio of the average yield point determined by the bending test to that determined by the direct tension test is 1.75. The corresponding average ratio for the 1-in. bars tested in the bending machine shown in Fig. 5 was 2.42. The agreement between these ratios is not close but it is sufficiently good to warrant the conclusion that the yield point determined from the bend test of a round bar may be expected to be markedly greater than that determined from direct tension. The difference between the ratios may be due partly to error in determining the pull in the bend test machine and partly to the fact that with that machine the load was applied almost instantaneously while in the screw-power testing machine it was applied very slowly.

The stresses computed from the bend tests employ the usual assumptions that the neutral axis is at the center of the bar and that the stress at any point is proportional to the distance of the point from the neutral axis. If at the yield point load on the bar in the bend test the neutral axis be still assumed to be the center of the depth but the stress be assumed to be at the yield point throughout the tension area the resisting moment would be 1.7 times as great as under the former assumption (linear variation of stress). This latter assumption is so extreme as to be untenable but it suggests the nature of an actual stress distribution, which would account for considerable excess of the apparent yield point stress computed from bend tests over that determined from direct tension.

Since these tests were carried out data on the relation between tensile resistance in direct tension and that in bending have been found which corroborate the data from the Bureau of Standards tests. A. B. W. Kennedy[1] reports that about thirty-five years previous to the date of his article (1923) he had made tests on cast iron in which the ratio of yield point strength computed from bend tests to the yield point strength from direct tension tests averaged 2.13 for round bars, and 2.11 for square bars on edge. For recent tests of square bars tested on edge the corresponding ratio for cast iron was 2.39 and for steel 1.98. No recent tests of round bars are reported. This corroboration of the results given in the preceding paragraphs is sufficiently good to make it seem certain that the high yield point determined from the bend tests and the small deflection in the impact tests are to be accounted for through the properties of the sections tested and not as an incorrect interpretation of the data secured.

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[1] A. B. W. Kennedy, "Experiments on the Yield Point of Steel in Transverse Tests," *Engineering* (London), June 15, 1923, p. 736.

IMPACT TESTS

The impact test was intended for the purpose of detecting brittleness in the reinforcement bars which might be so great as to cause breakage in the handling of bars on the job. The criterion for an impact suitable for this purpose was assumed to be a blow sufficient to stress the bar well beyond its yield point. The machine with the tup used for testing the 1-in. bars is shown in Fig. 8. The design was made to give similar stress conditions in specimens of different sizes. For the specimens to be equally stressed under the impact it is necessary: (1) that the ratio of the span to the diameter of the specimen be the same for all sizes of bars; (2) that the ratio of the mass of the tup to the mass of the specimen be constant; (3) that the height of fall be maintained constant. The span used was 24 times the diameter of the test specimens. The volume of the tup was 7.48 times the volume of that portion of the specimen between the supports. The height of fall was 30 in.

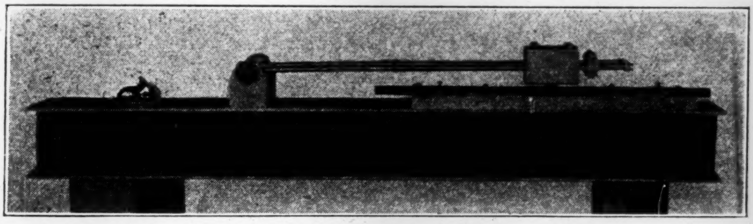

FIG. 8.—Impact Machine with Tup for Testing 1-in. Bars.

In order to fulfill strictly the requirement that the ratio of the mass of the tup to the mass of the specimen be constant, the length of that portion of the specimen overhanging the supports should have been a constant number of times the diameter of the specimen. This overhang ranged from six diameters with the ½-in. and the 1-in. bars to twelve diameters with the ⅜-in. and the ¾-in. bars. The indications from the test data are that this slight variation from the theoretical conditions did not have an important effect on the test results.

The design was made so that the center of mass of the falling system using any one of the four hammers required, would fall through 30 in. and so that the center of percussion of the system would be at the center of mass of the tup. This last condition was accomplished by using a small counterpoise as shown in Fig. 8.

The weight in pounds of that portion of the specimen between supports was 5.35 d^3, d being given in inches. The weight in pounds

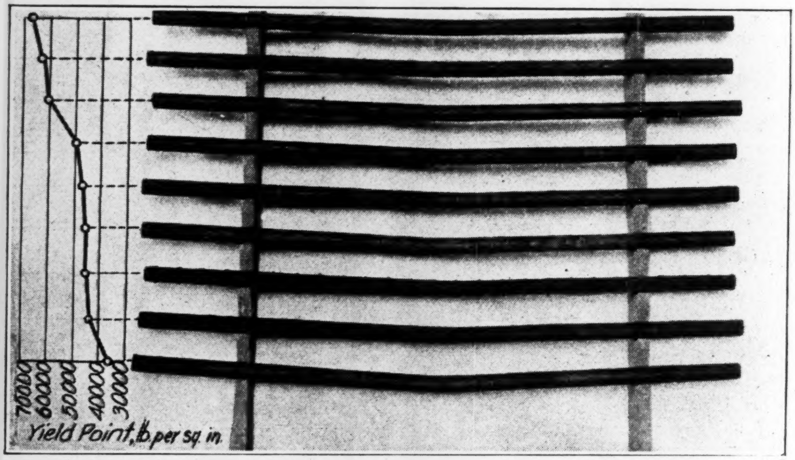

FIG. 9.—One-inch Bars After Impact Test.

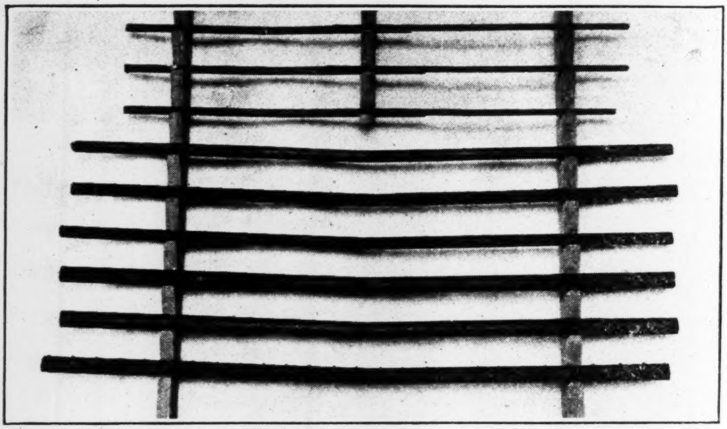

FIG. 10.—Bars of Four Sizes After Impact Test.

of the tup was 7.48 times as great, or 40 d^3. The energy of the hammer due to falling 30 in. was therefore, 1200 d^3 in-lb. Assuming that the elastic curve of the bar is the same as it would be if tested under static instead of impact load the elastic deflection would be $\frac{1}{48}\frac{Pl^3}{EI}$ and

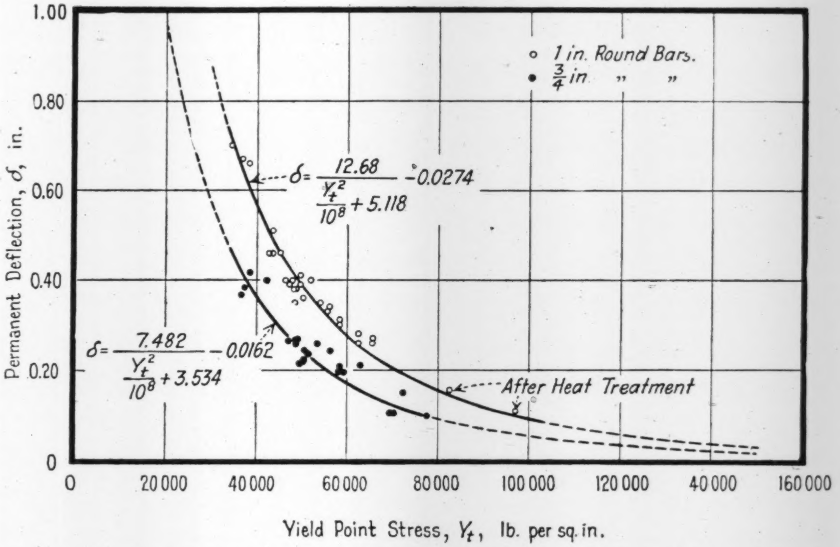

FIG. 11.—Relation of Yield Point to Permanent Deflection Under Impact for 1 and $\frac{3}{4}$-in. bars.

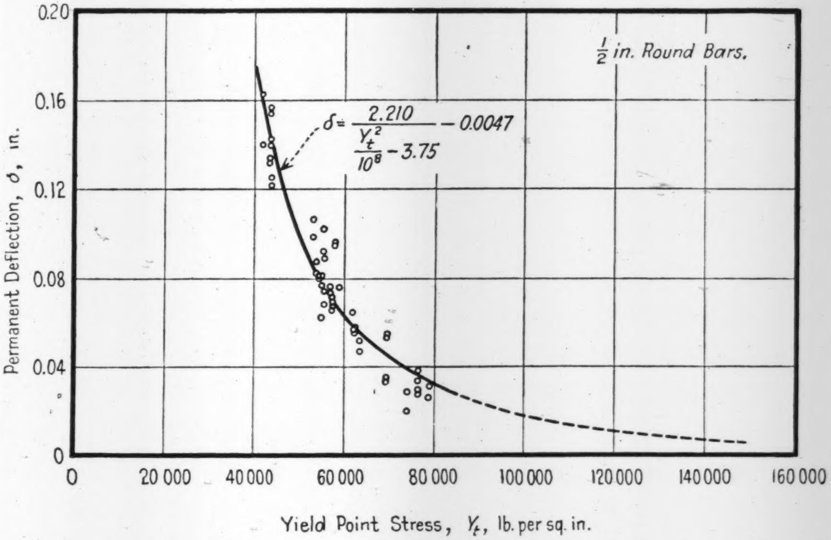

FIG. 12.—Relation of Yield Point to Permanent Deflection Under Impact for $\frac{1}{2}$-in. Bars.

the energy absorbed by the beam would be $\frac{1}{6}\,\frac{I}{(d/2)}\,\frac{2Sl}{E}$, where P, l, E, I and S are the center load, span, modulus of elasticity, moment of inertia, and stress, respectively, for the bar considered as a beam. Equating the energy in the tup to that absorbed by the beam, the stress S due to the blow is found to be 214,000 lb. per sq. in. Unless the yield point stress is greater than 214,000 lb. per sq. in. there should be a permanent deflection. Except as noted in the following paragraph all the bars were tested under the foregoing conditions.

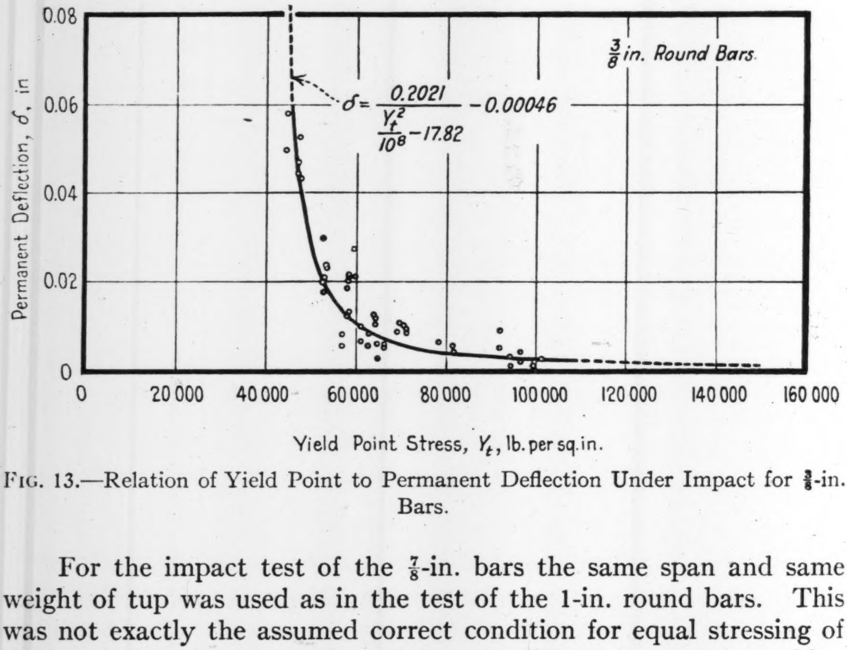

FIG. 13.—Relation of Yield Point to Permanent Deflection Under Impact for $\frac{3}{8}$-in. Bars.

For the impact test of the $\frac{7}{8}$-in. bars the same span and same weight of tup was used as in the test of the 1-in. round bars. This was not exactly the assumed correct condition for equal stressing of the bar, but is very near to it, and the tests did not warrant the making of a new tup. It will be observed that all the bars, although some of them could be bent cold through only small angles, had permanent deflections after the impact and that the lower yield points permitted a greater deflection.

None of the bars was so brittle as to break in the impact tests. Each showed a permanent bend at the point of application of the blow indicating that it was stressed beyond the yield point. In Fig. 9 are shown the A specimens from each lot of 1-in. bars arranged from the bottom to the top in order of increasing yield points determined in test No. 4. The decrease in deflection corresponding to the increase in yield point is here quite evident. In Fig. 10 are shown three bars of each size having yield points of approximately 55,000 lb. per sq. in. Except for the $\frac{3}{8}$-in. bars the angular deformation is about the same for the bars of different sizes. The amount of permanent deflection in the bars plotted against the yield point is shown in Figs. 11, 12, and 13. Generally when the yield point of a bar was small the permanent deflection was large. This deflection was roughly inversely proportional to the square of the yield point of the bar as is indicated by the form of the equations of the curves drawn through the points.

The curves through the points for the 1-in., $\frac{3}{4}$-in. and $\frac{1}{2}$-in. bars are given by the general equation:

$$\delta = \frac{20.95d - 8.265}{\frac{Y_t^2}{10^8} + 6.384 - \frac{1.267}{d^3}} - 0.0454d + 0.018$$

where δ = the permanent deflection in inches,
Y_t = yield point in direct tension in pounds per square inch, and
d = diameter of the specimen in inches.

The curve which was fitted to the data of the $\frac{3}{8}$-in. bars has the equation:

$$\delta = \frac{0.2021}{\frac{Y_t^2}{10^8} - 17.82} - 0.00046$$

While it might be possible to derive a simple empirical equation which would include the data of the $\frac{3}{8}$-in. bars it was found difficult to do so. The empirical equations for these curves indicate the manner in which the deflection varied with the diameter of the bar and the yield point of the steel.

An attempt was made to account for the permanent deflections found in the impact tests by means of a rational analysis using the fact that the energy of the blow was known, and the assumption that the form of the elastic curve was the same as that for a single static load applied at the center of the span. Such an analysis required a knowledge of the yield point stress in bending and the modulus of elasticity before, during and after the yield point stress is reached. From the analysis made, the curve of computed deflections against assumed yield point stresses was of the same general shape as the curves shown in Figs. 11, 12, and 13 but the computed deflections were far different from those shown in these figures. No reasonable change in the assumptions regarding the modulus of elasticity had an appreciable effect in bringing the computed deflections into agreement with the observed deflections, but making the assumption that the yield point stress in the impact test was twice as great as the yield point stress determined from the tension tests improved the agreement greatly. This confirms the previous indication that the apparent yield point stress for the bars in bending was considerably greater than the yield point stress determined by tension tests. No constant ratio between the yield points determined by the two methods was found, however, which when introduced into the analysis would give computed deflections which were in satisfactory agreement with the observed deflections. While the difference in apparent yield points was the greatest factor in the discrepancy, other factors evidently were too important to be neglected.

The primary purpose of the impact test was that of detecting brittleness, but from the consistency of the relation between deflection, yield point and diameter of bar it seems that the impact test might be used as a fairly satisfactory method for estimating the yield point of reinforcement bars. The usefulness of the test for this purpose is somewhat impaired, however, by the fact that it does not seem feasible to state an equation for permanent deflection which includes the results for the $\frac{3}{8}$-in. bars.

Since none of the bars subjected to the impact test broke under the blow it was not possible to know for a certainty that the test was severe enough to detect brittle specimens. In view of the fact that there were no available specimens of bars that had broken in handling, it was impossible to determine what the effect of the blow on such steel would be. In order that the necessary degree of brittleness be approximated one 1-in. specimen from each of the bars of lots B, D, E, F, G, H and J was treated by quenching in water when heated to 1600° F.

When subjected to the impact test the lot F bar broke in two places, one on each side of the center of the span. None of the other bars broke but they showed permanent deflection ranging from 0.100 in. to 0.155 in. The carbon content of these bars was from 0.45 per cent to 0.52 per cent. The lot F bar had a carbon content of 0.47 per cent. With the one exception the higher the carbon content for these heat-treated bars the less was the permanent deflection.

A further test of the severity of the impact was made on 1-in. specimens BC, DB, and EB by subjecting them, when supported as for 1-in. bars, to the blow given $\frac{3}{4}$-in. bars. Under this condition the bars showed permanent deflection of 0.039 in., 0.034 in., and 0.022 in., respectively. Since this lighter tup stressed these 1-in. bars beyond their yield points (as indicated by the permanent deflection) the same blow applied to the $\frac{3}{4}$-in. bars obviously was a severe test.

From the impact test on the heat-treated specimens and the test of the 1-in. bars with the hammer used for the $\frac{3}{4}$-in. specimens it would seem that the test is severe enough to detect brittleness and indicate bars likely to break in handling.

If all the bars having the same yield point had been stressed to the same intensity under the impact the permanent deflection should have been proportional to the span. That is, the angle of the bend should have been the same in all bars having equal yield points. In Fig. 10 showing different sizes of bars having nearly the same yield points it will be seen that except for the smallest bars ($\frac{3}{8}$ in.) the angle of permanent bend is nearly the same for all sizes. Undoubtedly all of them were stressed beyond their yield points. For the condition that the bars be not permanently bent by the blows the yield points due to bending would have to be greater than 214,000 lb. per sq. in. for all sizes of bars, the stress at which the energy of the hammer is balanced by the work of deformation in the bar under elastic conditions.

CONCLUSIONS

1. The tensile strength of the bars, generally, was only slightly affected by the previous bending of the bar. The amount of this effect was not materially dependent on the method used for bending the specimens.

2. The impact test gave an approximate measure of the yield point of a bar, and it is believed should detect the presence of extremely brittle bars having yield points within the range of practice.

3. The special bending machine seemed to give the results aimed at, that is, with its use all bars of the same yield point underwent about the same strain in bending regardless of their size.

4. While all but two of the specimens other than the $\frac{7}{8}$-in. square bars met the bend test regardless of the yield point, many of the $\frac{7}{8}$-in. bars failed in the bend test. Using these bars as a criterion it is important that for yield points above, say, 65,000 lb. per sq. in. special attention be given to the results of the bend test.

5. It was found that the apparent yield points in the bend test were much higher than the yield points determined in direct tension tests, being on the average approximately 2.4, 2.6, 2.8 and 3.0 times as high for bars 1, $\frac{3}{4}$, $\frac{1}{2}$ and $\frac{3}{8}$ in. in diameter, respectively.

Acknowledgment.—Acknowledgment is made to L. B. Tuckerman of the Bureau of Standards for his generous assistance in determining the principles to be adhered to in making the cold bend and impact tests.

A CRITICAL STUDY OF THE A.S.T.M. CORROSION DATA ON UNCOATED COMMERCIAL IRON AND STEEL SHEETS

By V. V. Kendall[1] and E. S. Taylerson[2]

Synopsis

The data resulting from the atmospheric and total immersion corrosion tests conducted by the Society's Committee A-5 on Corrosion of Iron and Steel have been analyzed by the statistical method to determine the effect on corrosion resistance of copper, phosphorus, manganese and sulfur and their combinations. The results are given in the form of curves. In the atmospheric test at Pittsburgh it was found that while copper is definitely beneficial, the longest life is obtained with high copper combined with phosphorus. Manganese and sulfur seem to have little, if any, effect either way. While the Fort Sheridan test confirms these conclusions as regards copper, it only partly confirms those in regard to the other elements due to the lack of complete data. In the total-immersion tests, none of these elements have a material effect on the rate of corrosion. Under these total-immersion conditions the effect of ordinary variations in composition is secondary to external influences.

The following study of corrosion data accumulated by the Society's Committee A-5 on Corrosion of Iron and Steel is not intended in any way to represent the views or conclusions of the Society or of the members of Committee A-5. The authors alone are responsible for the methods of analysis, the presentation of the data, and the conclusions expressed in this paper.

Introduction

The extreme difficulty of assigning any definite rôle to one particular element in commercial steels is not generally realized. To vary one element in steel and assign the results, whether beneficial or otherwise, to that element alone, without at the same time investigating the effect of variations in the amount of other elements is a procedure of questionable merit. The effect of combinations of elements is just as important as that of each individual constituent. It is seldom that sufficient specimens with varying ranges of the common elements are included in a corrosion test to enable the effect of each element to be evaluated. While the corrosion data accumulated by Committee A-5 in its large-scale corrosion tests on uncoated commercial steels are in this respect by no means complete, a considerable range of compositions is available. The authors studied these data to determine the separate and concurrent effects, if any, of copper, phosphorus, manganese, and sulfur in uncoated commercial irons and steels under atmospheric and submerged corrosion conditions. Daeves[1] in 1926 investigated the atmospheric tests by a graphical method and found a 50-per-cent longer life for the copper-bearing material. He did not, however, investigate the other elements. Space does not permit the discussion of the results of other investigations of copper and non-copper steels. Our attention here will be confined entirely to the published data of Committee A-5 in an attempt to determine the tendencies of certain elements to hinder or promote corrosion under the particular conditions of these tests.

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METHOD OF INVESTIGATION

In the study of a mass of data in which the variable factors are numerous, the most promising line of attack seems to be a statistical one involving the grouping and averaging of results in all possible combinations. It is, at best, laborious. However, the use of available tabulating machines has so decreased the work involved as to make this paper possible. The complete information for each specimen in the test is transferred to a card, by punching holes at certain locations, and in appropriate columns. The whole group of cards is then sorted electrically into the particular order desired and thus into groups which can be further subdivided if necessary. These groups of cards are then tabulated by a machine which automatically prints the figures from those columns which are of interest with totals for each group and the number of cards in each group. After averaging, the data are then ready to plot in the form of curves.

A.S.T.M. TESTS

The American Society for Testing Materials in 1916, through its Committee A-5, initiated a large-scale atmospheric test of commercial steel and iron, exposing large uncoated sheets of both Nos. 22 and 16 gage at three different locations. Pittsburgh (industrial atmosphere), Fort Sheridan (rural atmosphere), and Annapolis (seacoast atmosphere) were selected. At a later date (1920), small specimens from the same sheets were tested under total-immersion conditions in running city water at Washington; Calumet mine water at Pittsburgh, and

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[1] *Stahl und Eisen*, Vol. 46, p. 1857 (1926).

TABLE I.—SCHEDULE OF A.S.T.M. COMMITTEE A-5 CORROSION TESTS ON UNCOATED COMMERCIAL IRON AND STEEL SHEETS.

Location of Test	Duration of Test		Material			Failures to Date	
	Date Begun	Date Finished	Gage	Kind	Number of Sheets	Number	Per Cent
ATMOSPHERIC TESTS							
Pittsburgh, Pa.	Dec. 12, 1916	Discontinued Mar 9, 1923	No. 22	Copper.....	146	123	84.4
				Non-Copper.	84	84	100.0
				Total.....	230	207	90.0
			No. 16	Copper.....	132	none	0.0
				Non-Copper.	126	102	81.0
				Total.....	258	102	39.5
Ft. Sheridan, Ill.	Apr. 9, 1917	Discontinued Apr. 16, 1928	No. 22	Copper.....	136	50	36.8
				Non-Copper.	83	77	92.8
				Total.....	219	127	58.0
			No. 16	Copper.....	136	none	0.0
				Non-Copper.	124	4	3.2
				Total.....	260	4	1.5
Annapolis, Md.	Oct. 17, 1916	Last Inspection, May 7, 1928	No. 22	Copper.....	148	1	0.7
				Non-Copper.	79	23	29.2
				Total.....	227	24	10.6
			No. 16	Copper.....	130	none	0.0
				Non-Copper.	129	none	0.0
				Total.....	259	none	0.0
TOTAL IMMERSION TESTS							
Washington, D. C. Running City Water	Aug. 10, 1920	Oct. 12, 1923	No. 22	Copper.....	84	All	100.0
				Non-Copper	60	All	100.0
				Total.....	144	All	100.0
		Jan. 19, 1928	No. 16	Copper.....	84	All	100.0
				Non-Copper.	66	All	100.0
				Total.....	150	All	100.0
Annapolis, Md. Severn River Water. Brackish Water.	Aug. 10, 1920	Oct. 12, 1923	No. 22	Copper.....	78	All	100.0
				Non-Copper.	66	All	100.0
				Total.....	144	All	100.0
	Jan. 2, 1920	Last Isnpection Feb. 11, 1928	No. 16	Copper.....	84	70	83.3
				Non-Copper.	66	62	94.0
				Total.....	150	132	88.0
Pittsburgh, Pa. Calumet Mine Acid Mine Water.	Aug. 6, 1920	Oct. 22, 1920	No. 22	Copper.....	90	All	100.0
				Non-Copper.	54	All	100.0
				Total.....	144	All	100.0
		Jan. 20, 1921	No. 16	Copper.....	84	All	100.0
				Non-Copper.	66	All	100.0
				Total.....	150	All	100.0

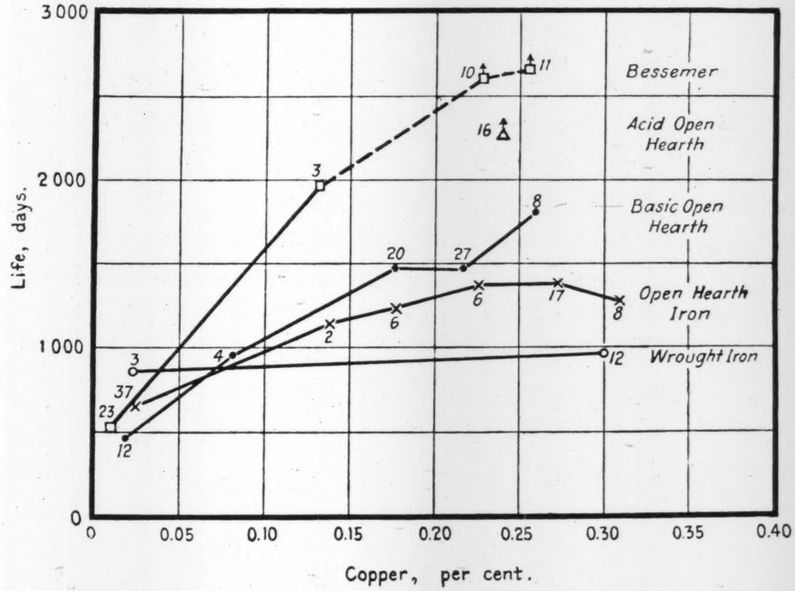

FIG. 1.—Pittsburgh Tests, No. 22 Gage Sheets—Showing Relation of Copper to Life in Days on Various Types of Steel.

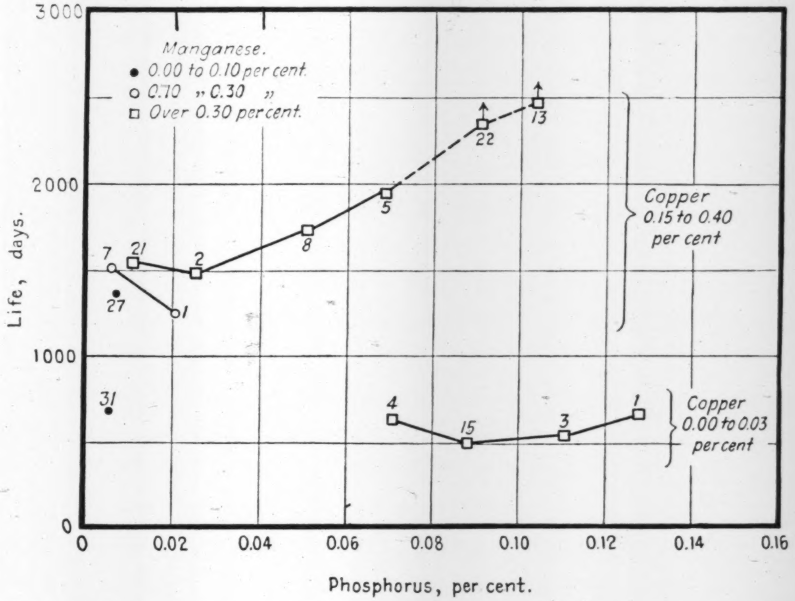

FIG. 2.—Pittsburgh Tests, No. 22 Gage Sheets—Showing Effect of Phosphorus, Copper and Manganese.

Severn River brackish water at Annapolis. For details of these tests see the reports of Committee A-5 for the periods given in Table I.

From this table it will be seen that of the atmospheric corrosion tests only the Pittsburgh No. 22 gage and the Fort Sheridan No. 22 gage are sufficiently complete for the purpose of this investigation. All of the total-immersion tests can be studied, although the Annapolis No. 16 gage can be studied only to a limited degree.

PITTSBURGH NO. 22 GAGE ATMOSPHERIC TEST

In this test 10 per cent (23 out of 230) of the sheets had not failed. For purposes of calculation 10 months was the arbitrary figure added to the life of these sheets (and the Fort Sheridan sheets also) and the points affected marked by an arrow pointing upward.

The sheets were first divided into types of steel and then into groups of 0.05 per cent copper. The life of each of these groups is shown in Fig. 1. The number at each point is the number of sheets involved in the average figure represented by this point. Inspections were made at approximately 6-month or 180-day intervals, which is approximately one-fifth of the numbered scale divisions.

The curves in Fig. 1 demonstrate that under these conditions of exposure, the addition of copper has no noteworthy effect on the corrosion resistance of wrought iron, a definitely beneficial effect on open-hearth iron and steel, which beneficial effect is even more pronounced in the case of bessemer steel. Acid open-hearth steel containing copper was also exceptionally good. There was no non-copper acid open-hearth steel present in the test. It also shows quite definitely that copper is not the only controlling factor and that the maximum effect of copper is only obtained in combination with other elements.

The lives of the sheets, arranged in groups of 0.05 per cent of copper, were then plotted against phosphorus content, taking copper and manganese into consideration. The results are shown in Fig. 2[1]

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[1] With reference to Figs. 2, 3, 6, 8, 9, 13, 14, 15, and 16, in studying the effects of copper, manganese and phosphorus, respectively, where only one element is considered, all the sheets involved in the tests have been included. It would obviously be illogical to study the effects of phosphorus in the case of the wrought-iron sheets, as a large percentage of the phosphorus is combined with the slag inclusions, and can have little effect as an alloying element in the metal. (See Fig. 5, points 1*, 3*, 8*, and 3*, which refer to wrought iron.) Some of the No. 22 gage basic open-hearth sheets were abnormally high in silicon, and are not representative of regular basic open-hearth steel low in copper. For these reasons, the wrought iron ("E" and "Y" groups) and the high silicon ("O" groups) were excluded in the detailed study of more than one element at a time.

To simplify this study, the copper groups were confined to those very low in copper (zero to 0.03 per cent), and to those from 0.15 to 0.40 per cent. This excluded a few sheets of intermediate copper content (0.03 to 0.15 per cent) and a few miscellaneous "Z" sheets of high copper content (over 0.40 per cent).

In the case of the points involving one or more sheets that had not failed at the conclusion of the test, arrows indicate the trend of the life in days, and in some cases the curves are shown as an interrupted line to indicate this fact.

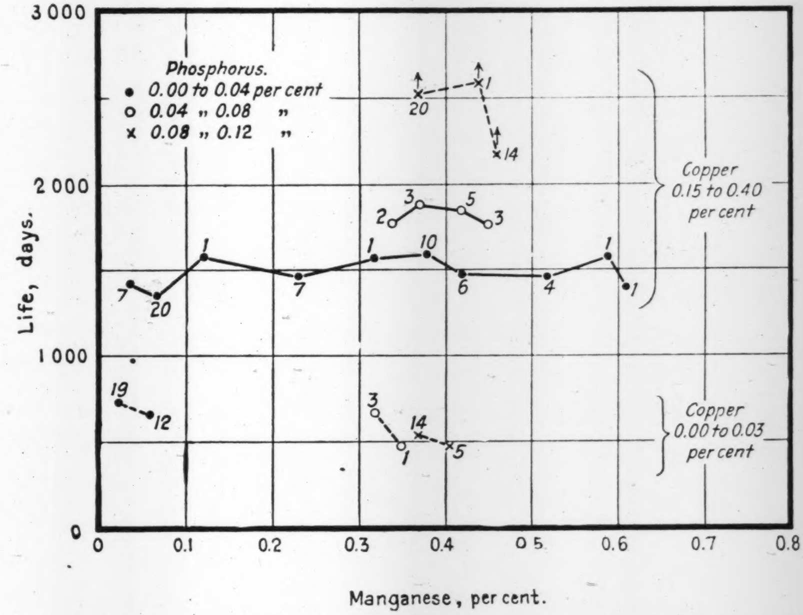

FIG. 3.—Pittsburgh Tests, No. 22 Gage Sheets—Showing Effect of Manganese, Copper and Phosphorus.

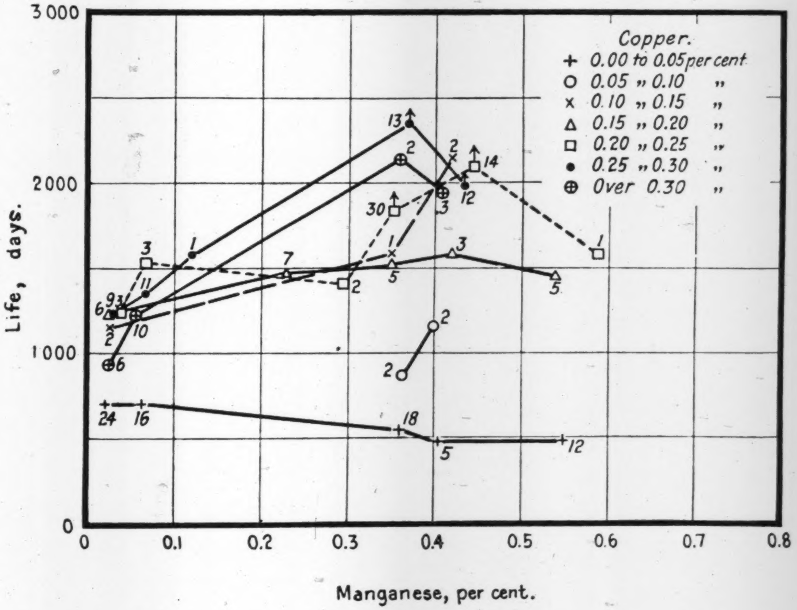

FIG. 4.—Pittsburgh Tests, No. 22 Gage Sheets—Showing Effect of Manganese for Various Copper Contents.

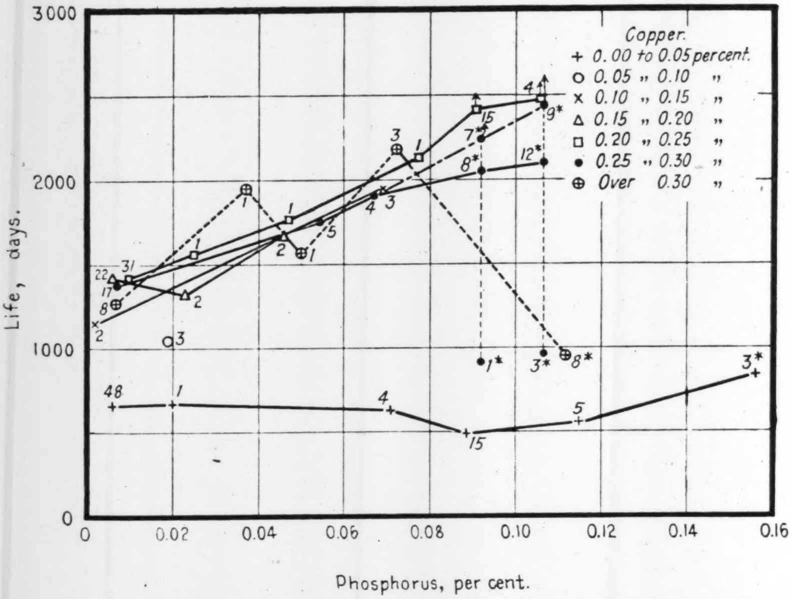

FIG. 5.—Pittsburgh Tests, No. 22 Gage Sheets—Showing Effect of Phosphorus for Various Copper Contents.

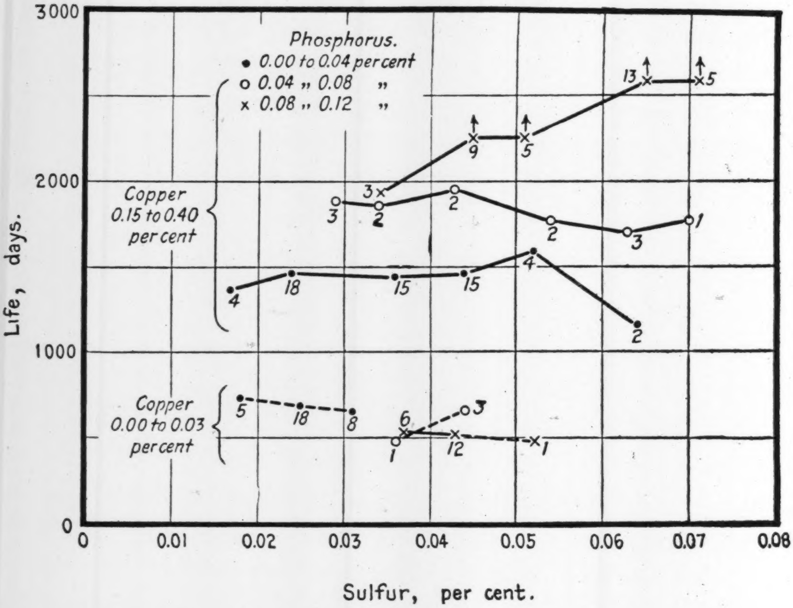

FIG. 6.—Pittsburgh Tests, No. 22 Gage Sheets—Showing Effect of Sulfur and Phosphorus.

and indicate a beneficial effect of phosphorus when combined with copper. Figure 3 shows, in a similar manner, the effect of manganese, which is seen to be neutral.

Figures 4 and 5 are inserted at this place to show the danger of drawing erroneous conclusions when all the significant factors are not considered. Figure 4 contains all of the sheets and shows an apparent beneficial effect of manganese. However, in this figure phosphorus is ignored; when this factor is taken care of, Fig. 3 is obtained and shows that the apparent effect of manganese was in reality due to phosphorus and that manganese has little if any effect. The wrought irons in Fig. 4 are in the low-manganese points and have a tendency to pull down the higher copper points. Figure 5 also contains all of the sheets plotted against phosphorus but ignoring manganese. The effect of the wrought irons is very decidedly shown in this figure. The points marked with an asterisk are the points affected by the wrought irons, the wrought-iron sheets being the ones numbered 1*, 3*, 8*, 3*. The point 12* in the 0.25 to 0.30-per-cent copper group contains 3 wrought-iron sheets and 9 of other material; point 8* in the same group contains one wrought iron and 7 of other material. When these are separated, the points are raised to a higher position at 9* and 7*. In the high-copper (over 0.30 per cent) curve, the final point (8*) represents that number of wrought-iron sheets. When all of the significant factors in Fig. 5 are considered, Fig. 2 is obtained and, in this case, the tendency shown is confirmed. It is absolutely necessary in a study of this kind not to overlook any significant factor.

Sulfur has often been considered deleterious, with little definite evidence in support of that conclusion. The sheets divided into low- and high-copper groups, each in ranges of phosphorus, are plotted against sulfur in Fig. 6. Sulfur does not appear to have an appreciable detrimental effect. One curve, high copper, 0.08 to 0.12 per cent phosphorus, has a definite upward trend and four of the high points contain sheets that had not failed. High copper combined with high phosphorus apparently counteracts any bad effect due to sulfur. In the low-copper group, high sulfur and high phosphorus, within the limits studied, have little influence.

FORT SHERIDAN NO. 22 GAGE ATMOSPHERIC TEST

The Fort Sheridan test was necessarily discontinued after only 58 per cent of the sheets had failed. It is admittedly difficult to draw other than general conclusions which, with a few exceptions, merely confirm the results of the Pittsburgh No. 22 gage test. The real

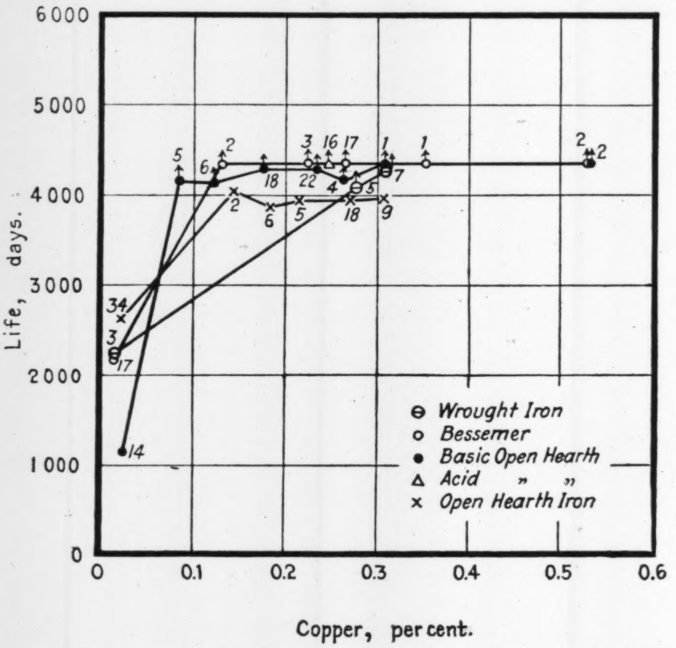

FIG. 7.—Fort Sheridan Tests, No. 22 Gage Sheets—Showing Relation of Copper to Life in Days on Various Types of Steel and Iron.

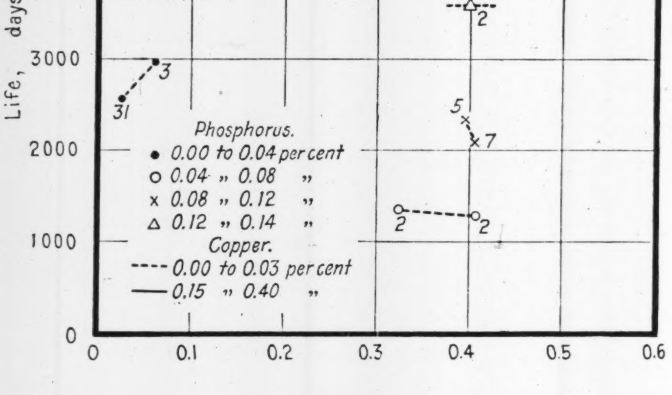

FIG. 8.—Fort Sheridan Tests, No. 22 Gage Sheets—Showing Effect of Manganese, Copper and Phosphorus.

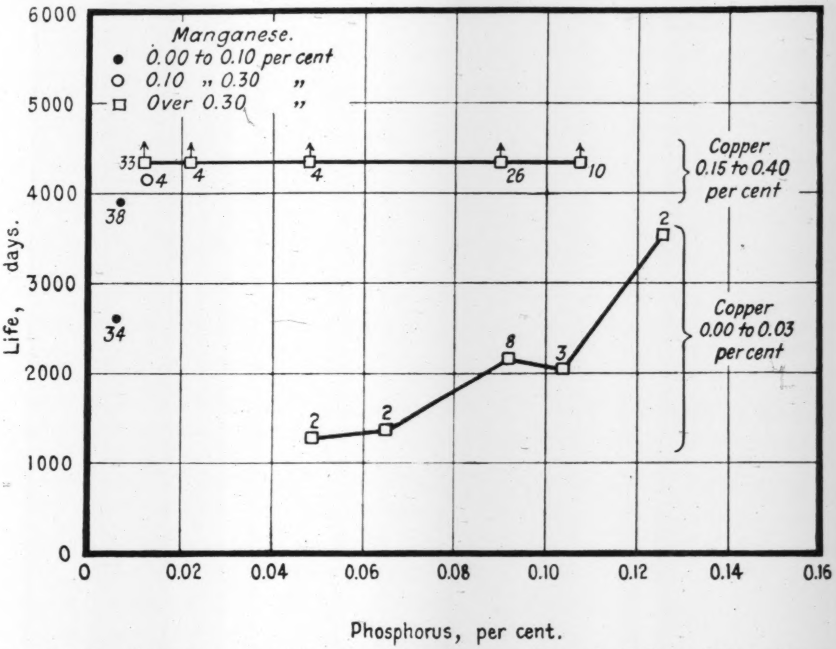

FIG. 9.—Fort Sheridan Tests, No. 22 Gage Sheets—Showing Effect of Phosphorus, Copper and Manganese.

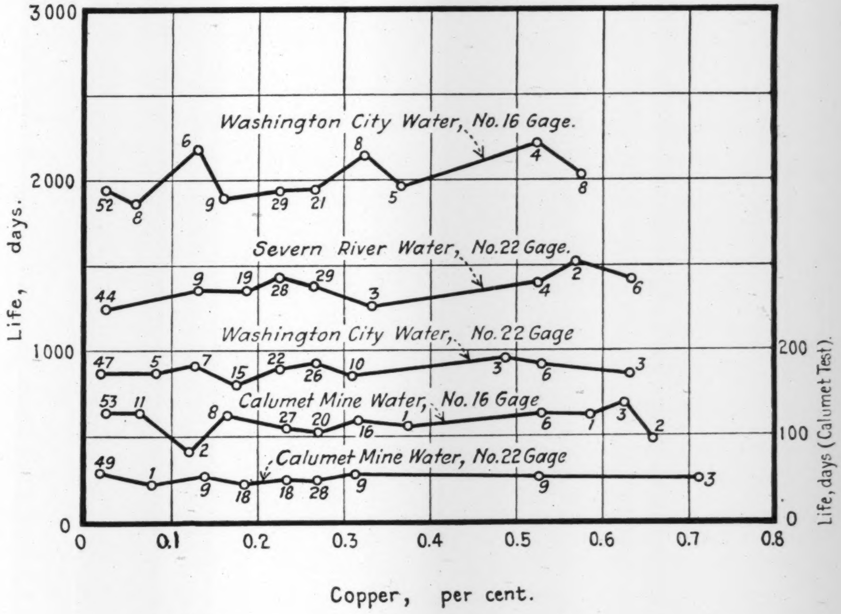

FIG. 10.—Total Immersion Tests—Showing Effect of Copper.

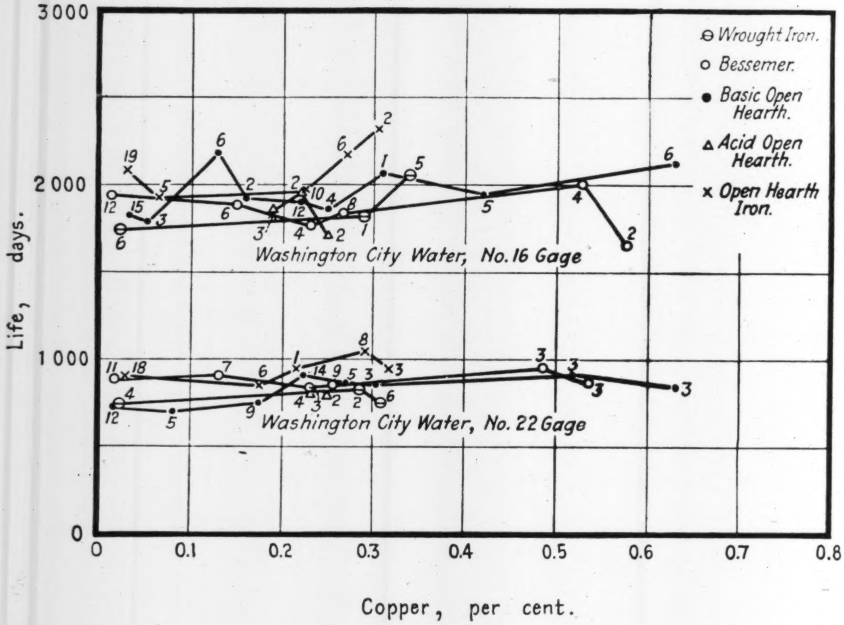

FIG. 11.—Total Immersion Tests—Showing Relation of Copper to Life in Days on Various Types of Steel and Iron.

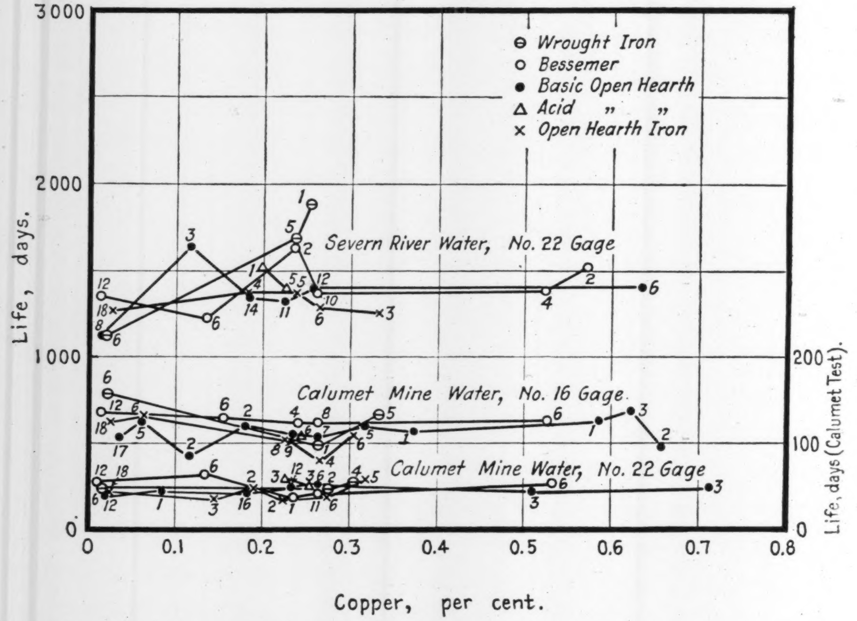

FIG. 12.—Total Immersion Tests—Showing Relation of Copper to Life in Days on Various Types of Steel and Iron.

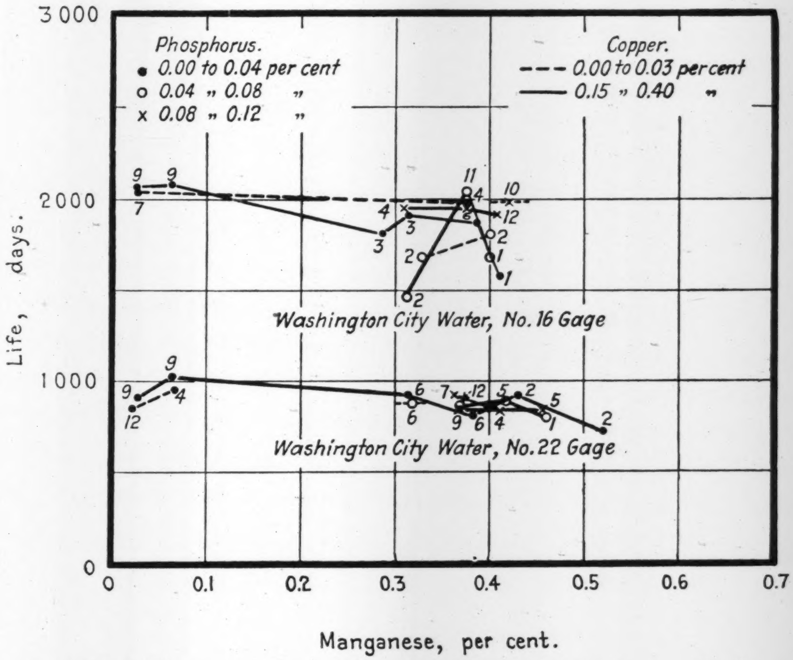

FIG. 13.—Total Immersion Tests—Showing Effect of Manganese, Copper and Phosphorus.

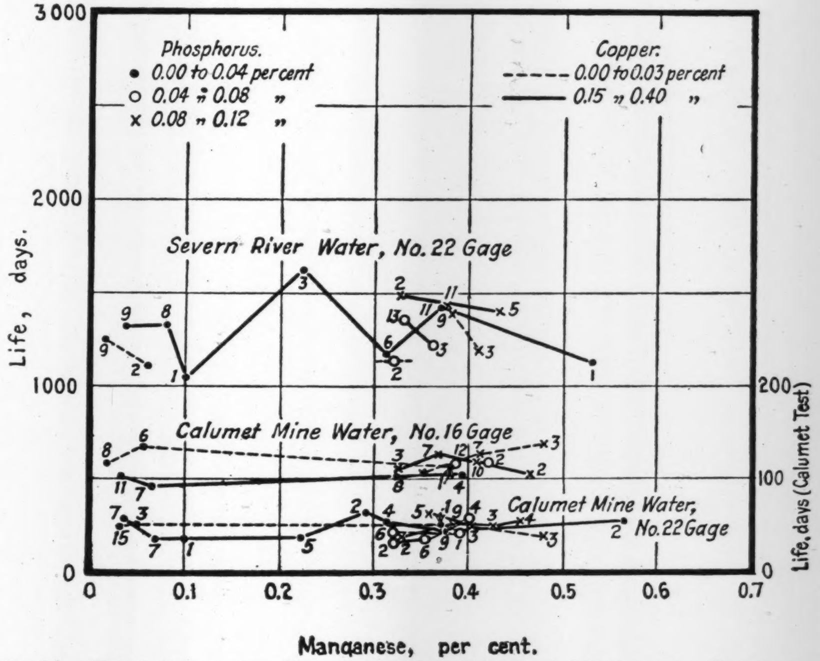

FIG. 14.—Total Immersion Tests—Showing Effect of Manganese, Copper and Phosphorus.

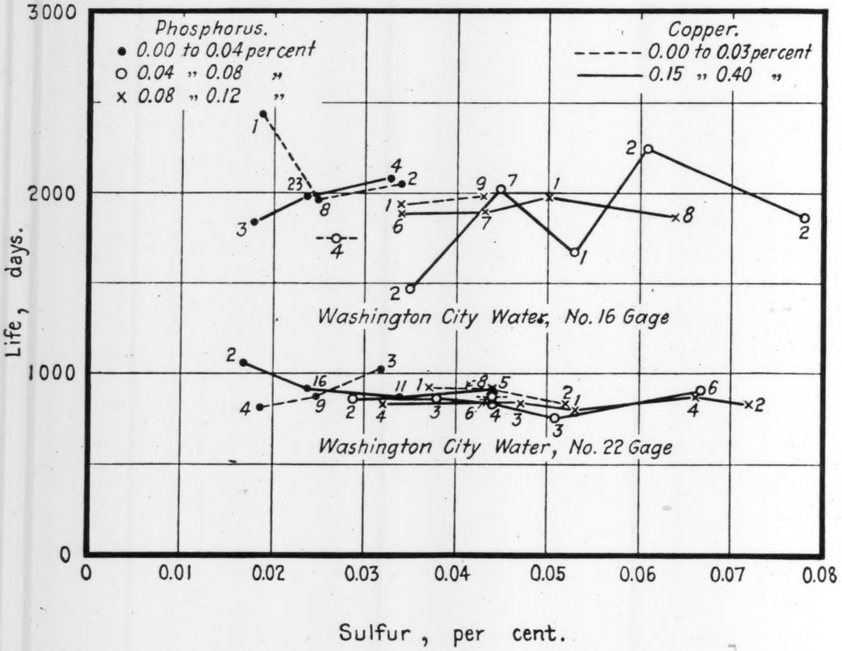

FIG. 15.—Total Immersion Tests—Showing Effect of Sulfur, Copper and Phosphorus.

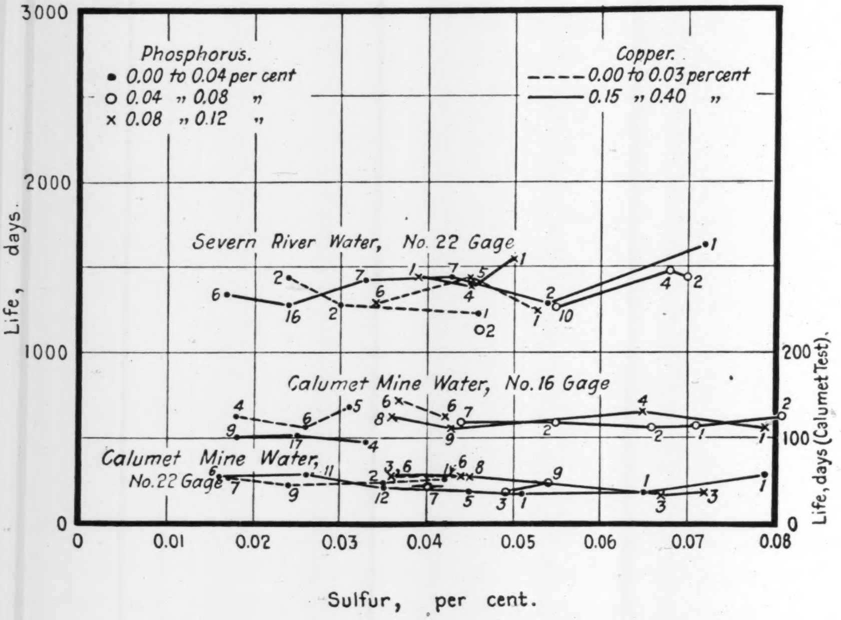

FIG. 16.—Total Immersion Tests—Showing Effect of Sulfur, Copper and Phosphorus.

information on the elements other than copper is in the 42 per cent that did not fail. Nevertheless, there are certain facts that are clearly brought out by a study of this test. Figure 7 shows that the effect of copper was decidedly beneficial for all types of material. Notwithstanding this evident beneficial effect, all of the open-hearth iron sheets were classified as failed at the end of the test.

The procedure followed in the study of the Pittsburgh data was also followed in this case. In Fig. 8, the low- and high-copper groups are sub-divided according to phosphorus content. The effect of manganese on the life can thus be shown irrespective of these other variables. In the low-copper groups, little chance for comparison is afforded on account of the lack of enough variation in analysis.

In Fig. 9, the low- and high-copper groups, sub-divided into manganese ranges (zero to 0.10 per cent and above 0.30 per cent) are studied to show the relation of the life as affected by the phosphorus content. In the high-manganese low-copper group, phosphorus seems to have a somewhat beneficial effect.

TOTAL IMMERSION TESTS

In the detailed study of the immersion tests, a procedure was followed similar to that used in the atmospheric tests.

Figures 10, 11 and 12 give the life in days plotted against copper content (groups of 0.05 per cent), both for the whole series of sheets and also classified according to type of iron or steel.

Figures 13, 14, 15, and 16 are studies of the effects of manganese, phosphorus, and sulfur variation in low (zero to 0.03 per cent) and high (0.15 to 0.40 per cent) copper groups respectively. The indications furnished by this group of curves are so erratic that they cannot be used safely in drawing conclusions as to the effect of any particular constituent.

Some students of ferrous corrosion under immersion conditions have come to the conclusion that composition of commercial steel or iron is much less important than external conditions, such as oxygen supply and type of water. If this theory is correct, the results of the immersion tests should obey the laws of chance, and should conform to the probability curve.[1]

A number of probability curves, based on the whole groups and sub-divisions indicated that these results do seem to be governed mainly by external conditions.

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[1] The invention of probability paper by Hazen, *Transactions*, Am. Soc. Civil Engrs., Vol. 77, p. 1939 (1914), has simplified the employment of this method of study. The horizontal scale is symmetrically distorted in proportion to the probability integral so that a straight line will result if the data are subject to the normal laws of chance.

The slope of the probability curve represents the degree of deviation of the results from the average.

In these immersion tests the average life varies to a very great extent. Probability curves have been plotted in Fig. 17[1] in such a way that the slopes can be directly compared. The life is plotted on a scale which is different for each immersion condition but is, in each case, in terms of percentage of the average life of the group. Thus 10 per cent of the average life, shown to scale at the bottom of the figure, is of the same magnitude for all curves. The vertical positions of the curves have no significance.

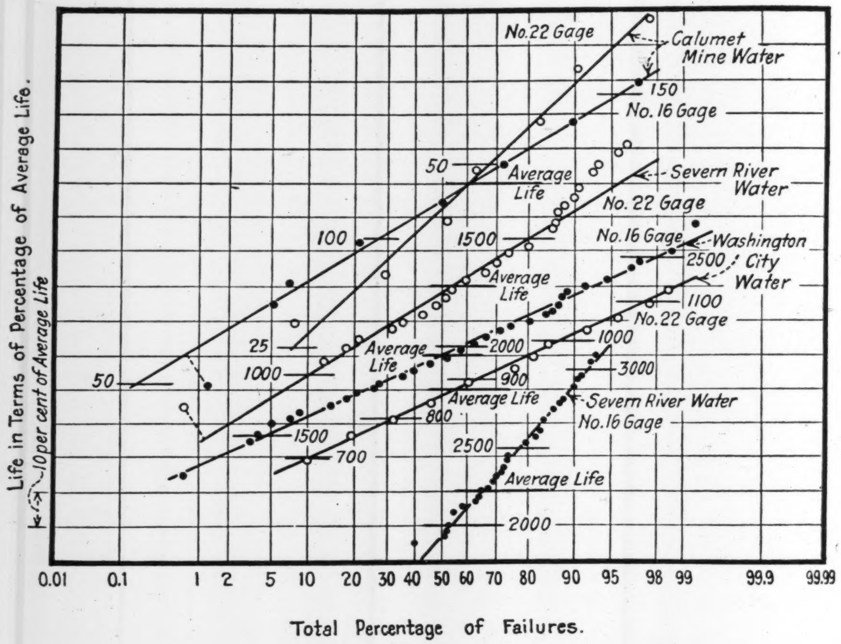

FIG. 17.—Probability Curves—Total Immersion Tests.

An inspection of these results will show that in most cases the points conform very closely to straight lines, although the No. 22 gage Annapolis Severn River data depart considerably from a straight line toward the end of the test, which may indicate the presence of some abnormal factor.

The intersection of the probability curve with the 50-per-cent coordinate represents the most probable average life of the group.

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[1] In the case of the Severn River No. 16 gage data, which are only 93 per cent complete, it was necessary to calculate the average life, assuming that the ten surviving sheets failed on March 30, 1928. This, however, is an unimportant assumption, as the average life of this test will be affected by the fact that 40 per cent of the sheets were perforated before the first inspection.

It is noteworthy that in every case this value is less than the observed average life. It is suggested that this is due to the fact that the sheets actually failed between inspections, while the records indicate the life at the time of examination.

The only conclusion that can be drawn from our investigation of the immersion tests is that the failures conform to the law of probability to a remarkable extent. This indicates that variations in external conditions, such as oxygen concentration, mill scale, location in the test tank, and adherence of corrosion products, which may initiate pitting in an entirely accidental manner, are the controlling factors in these tests.

SUMMARY

This study of the Pittsburgh and Fort Sheridan results confirms previous conclusions of Committee A-5 as to the effect of copper:

> At the close of eleven years' observations of these tests the sub-committee would direct attention to the fact that the failures at the Fort Sheridan location confirm the findings at the Pittsburgh location that copper-bearing metal shows marked superiority in rust resisting properties as compared to non-copper-bearing metal of substantially the same general composition under atmospheric exposure.[1]

It further brings out that at the Pittsburgh location, phosphorus combined with copper has very beneficial effects from an atmospheric corrosion standpoint. Manganese and sulfur, within the limits of composition involved, seem to have little if any effect either way.

The Fort Sheridan data, although limited on account of the incompleteness of the test, show similar beneficial results for high-copper material.

The total-immersion tests have been very thoroughly studied and, contrary to the atmospheric tests, the results indicate that the variation of the common elements, either singly or in combination, within the range of composition and under the conditions of these tests, has no material influence on corrosion resistance and is entirely secondary to external conditions.

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[1] Report of Committee A-5 on Corrosion of Iron and Steel, *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 159 (1928).

[For Discussion on Analysis of Corrosion Data, see page 232.—ED.]

A METHOD OF TREATING DATA ON THE LIVES OF FERROUS MATERIALS

By R. F. Passano[1] and Anson Hayes[2]

Synopsis

By the law of errors, the famed irregularity of corrosion test data becomes orthodox. The immersion tests of the Society's Committee A-5 on Corrosion of Iron and Steel have been plotted on Hazen's arithmetic probability paper. Characteristics of these curves are discussed in terms of the equation of the distribution curve, which has been modified for convenience. The service given by a material should be defined in terms of the "most probable life" and the spread in time of failure.

It is scarcely necessary to remark that the complicating feature in any numerical evidence is the group of unknown variables. Two persons born under similar circumstances at the same time do not acquire the same age before death. Two tension test specimens cut from the same bar tested in the same machine may not give exactly the same results. Material ordered to a given analysis, while essentially the same, is not always identical from one lot to another. Sheets ordered to a definite gage will vary above and below the ordered thickness. When two tanks stand side by side and are considered under the same condition, it has been observed that the one may serve twice as long as the other for no obvious reason. Pipe lines crossing the same territory may show radically different lives. It may so happen that something can be discovered about the two that is dissimilar, but is the observed variation in performance a measure of the dissimilarity?

Apparently even when every attempt is made to control the conditions, there is still a variation in the service given. Life insurance companies thrive in spite of the uncertainty of life, because of the certainty of the behavior of large numbers. Their problem is one of the distribution of lives and it is generally regular regardless of the multitudinous causes of death.

There is nothing new about the general principles of distribution. Gauss in 1809 deduced the equation of the probability curve or the law of errors in the form:

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[1] Research Engineer, The American Rolling Mill Co., Middletown, Ohio.

[2] Chief Chemist, The American Rolling Mill Co., Middletown, Ohio.

$$y = ke^{-h^2x^2} \tag{1}$$

in which form the curve is symmetrical about the Y axis.

In Fig. 1 is shown the result of plotting data obtained from firing 1000 shots from a field piece at a target 200 yards away which was divided into 11 horizontal strips. The number of shots in each strip is plotted against the position of that strip. The actual distribution is shown by the dotted curve. It differs from the theoretical distribution curve (shown by the solid line) probably because the force of gravity was a constant error for which no compensation had been made in the sighting. This graph is included to show that both positive and negative errors occur in about the same proportion and that these are distributed about the mark at which each shot was aimed.

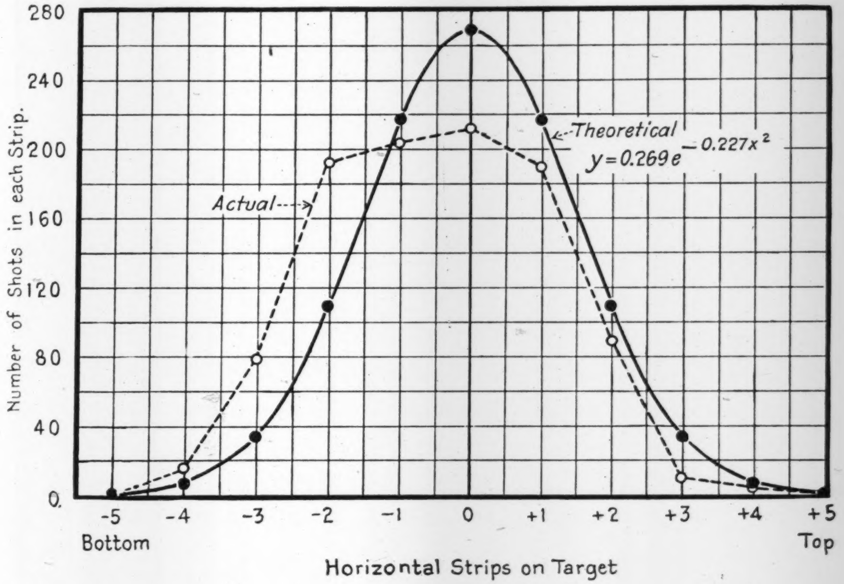

FIG. 1.—Distribution of Shots on a Target. (From 1878 Report of Chief of Ordnance.)

When dealing with errors, one thinks in terms of a value with deviations both plus and minus. Thought of life however is generally restricted to a finite figure or to a range of positive numbers. For this case, Eq. 1 may be modified to the form:

$$y = ke^{-h^2(a-x)^2} \tag{2}$$

in which y is the fraction which fails at a given time, x is time in the same unit as a, and h is a constant.

Differentiating,

$$y' = -2h^2ke^{-h^2(a-x)^2} \cdot (a-x) \quad \ldots\ldots\ldots\ldots\ldots (3)$$

In the case of a maximum or minimum

$$y' = 0$$

or, in this case

$$-2h^2ke^{-h^2(a-x)^2} \cdot (a-x) = 0 \quad \ldots\ldots\ldots\ldots\ldots (4)$$

When $(a - x) = 0$

$$x = a$$

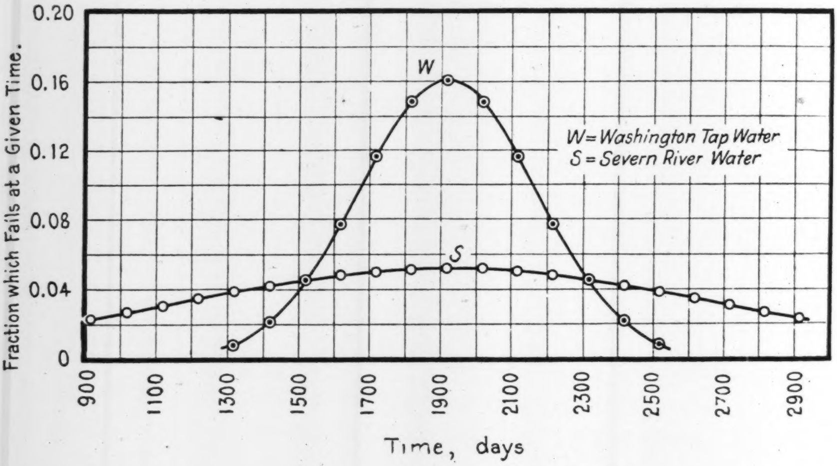

FIG. 2.—Calculated Distribution Curves of Failures of No. 16 Gage Iron and Steel Sheets in Washington Tap Water and Severn River Water.

Since the curve of Eq. 2 has no minimum, the conditions establish that there is a maximum at the point where $x = a$, from which a is the most probable life.

Returning to Eq. 2 and setting $x = a$, the equation reduces to the form

$$y = ke^0$$

or

$$y = k$$

so that k then represents the fraction which fails at the most probable life, a.

It sounds foolish to say that a definite number of samples can fail at a single instant, which is the exact theoretical significance of calculated values. In spite of this apparent fallaciousness there are certain characteristics of the distribution curve which are of considerable importance.

In Fig. 2 are shown two calculated distribution curves for failures of No. 16 gage iron and steel sheets in Washington tap water and Severn River water. In case "W" (Washington tap water), the failures occur over a narrow range, whereas in case "S" there is a larger spread,[1] although the most probable life of the two conditions is essentially the same. It is apparent that an impression of short life, or undesirable material, might be gained from the early failures in condition "S." The form of distribution curve "W" is, then, considered more desirable than the form "S."

Practically, the number which fail at a given time depends on the difference between the number which had failed when the series was last examined and the number which are now known to be failures. Thus the fraction which fail at a given time represents a summation of individual events over the inspection interval.

The only guides which can be used to determine the inspection interval are in those fields where previous experience on structures is available. In these cases, the inspection period should be chosen such that the number of failures which occur in each interval are of considerable magnitude. For, if the inspection interval is so short that only one or two failures are found, then variation in the behavior of small numbers will cause major irregularities in the distribution curve. It can be seen that the time between inspections should vary in accordance with the manner in which failures occur, both with respect to their most probable life and distribution about that value. Where there is no previous experience, the inspection interval may have been influenced by the weather conditions or some unrelated bit of humor or prejudice. All of these difficulties may be overcome by avoiding temporarily the distribution curve itself in favor of a treatment which would include all failures up to a given time. Mathematically, the notion of summation signifies integration, which process is indicated in Eq. 5:

$$T = k \int_0^x e^{-h^2(a-x)^2} . \, dx \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Unfortunately, this equation cannot be handled directly, although its value may be determined by actual measurement of area under the distribution curve, or by integration of a convergent series obtained by expansion of the expression within the integral sign. Fortunately, values of this integral are available in tables of physical constants.[1]

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[1] While this material was not inspected over its early life, values are shown which were calculated from distribution of failures beyond the most probable life.

It was Hazen who conceived the idea of constructing a variety of graph paper distorted on one axis such that the integral of the probability function would plot as a straight line. In a study entitled, "Storage to be Provided in Impounding Reservoirs for Municipal Water Supply,"[2] Hazen describes his paper as follows:

> "The spacing of the lines for this paper was computed from figures taken from the probability curve tables, and arranged so that the line which represents the summation of the probability curve, when plotted on it, is straight. If the data approximate the normal law of error, the line through the points will approximate a straight line.
>
> " . . . the 10-per-cent line is the same distance from the center on one side as the 90-per-cent line is on the other. Then 10 per cent of all the results will be outside of this line on each end, making 20 per cent in all. On page 221 of Merriman's "Least Squares" is found a table of values of the probability integral . . . In this table the value of x/r (representing the ratio of the actual variation to the probable error) corresponding to 0.8000 (80 per cent) is found to be 1.9. This figure, therefore, may be taken as the distance of the 10-per-cent line and the 90-per-cent line from the center of the diagram. The relative positions of other lines are found in the same way. The relative values only are of importance. They may be plotted on any convenient scale . . . "

Before we were aware that Hazen's paper was generally available, we had used distribution curves in studies of corrosion, physical data, composition of heats, etc., and appreciated that excessive numbers were necessary to produce smooth curves. Saklatwalla and Chandler[3] in the field of applied metallurgy have found probability methods of value. Taylerson in a discussion of Saklatwallas's paper comments on Hazen's graph sheet and was the first, so far as we know, to use such methods in problems of iron and steel manufacture. Griswold[4] has suggested distribution principles as a means of studying life of buried pipe. Speller has treated corrosion data with this special ruling prior to our activities, since he showed at a meeting of Sub-Committee V on Total Immersion Tests of the Society's Committee A-5 on Corrosion of Iron and Steel in June of 1928 a record of the No. 16 gage failures in Washington tap water. Kendall[5] has discussed a method of obtaining "average life" by use of the fundamentals of probability and this same arithmetic probability paper.

We have used the term "most probable life" instead of "average life" since it is a more accurate description of the significance of the value. On this one point our treatment coincides with that of Kendall, though the details in both cases are the results of independent thinking.

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[1] See Smithsonian Tables, Merriman's "Least Squares," etc.

[2] *Transactions*, Am. Soc. Civil Engrs., Vol. 77, p. 1539 (1914).

[3] *Transactions*, Am. Soc. Steel Treating, Vol. 10, p. 195 (1926).

[4] *American Gas Assn. Monthly*, March, 1928.

[5] *American Gas Assn. Monthly*, August, 1928.

There is no claim that either the graph paper or the type of material which can be handled on it, is unusual. It is, however, our belief that there are certain interesting things which can be derived from treatment of data by this method, which are not to be found in the present literature on the subject.

The corrosion of ferrous metals is a complicated situation. The fundamental reaction involved was shown by Whitney to be

$$Fe + 2H^{+} \longrightarrow Fe^{++} + H_2 \qquad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

the e. m. f. of which is given by the expression

$$E = \bar{E} - \frac{RT}{2\,F}\, ln \frac{(H^{+})^2}{(Fe^{++})\ .\ p \text{ of } H_2} \qquad \ldots\ldots\ldots\ldots\ldots (7)$$

which is the familiar form of expressing electrode potentials at various ion concentrations. $\bar{E}$ is the molal electrode potential, R is the familiar gas constant, T is the absolute temperature and F is 96,500 coulombs.

Some of the factors which may influence its value and the life of sheets are enumerated as follows:

(Fe^{++}),
(H^{+}),
Oxygen concentration,
Pressure of H_2,
Temperature,
Oxidation of Fe^{++} to Fe^{+++},
Composition and amount of salts in solution,
Variation in nature of cathode and anode,
Nature of compounds formed,
Influence of organic growths,
Composition of metal,
Variation in gage, and
Variation in thickness of coating.

It might be imagined that under a given set of conditions many of these factors would be constant. Such is no doubt true. However, the manner in which the corrosion begins and its distribution over the surface of the specimens under test are variable and complicate the determination of life. Thus there is reason to treat the lives of ferrous metal structures as a problem influenced by many variables, some of which can not be isolated. To such conditions does the probability method lend itself.

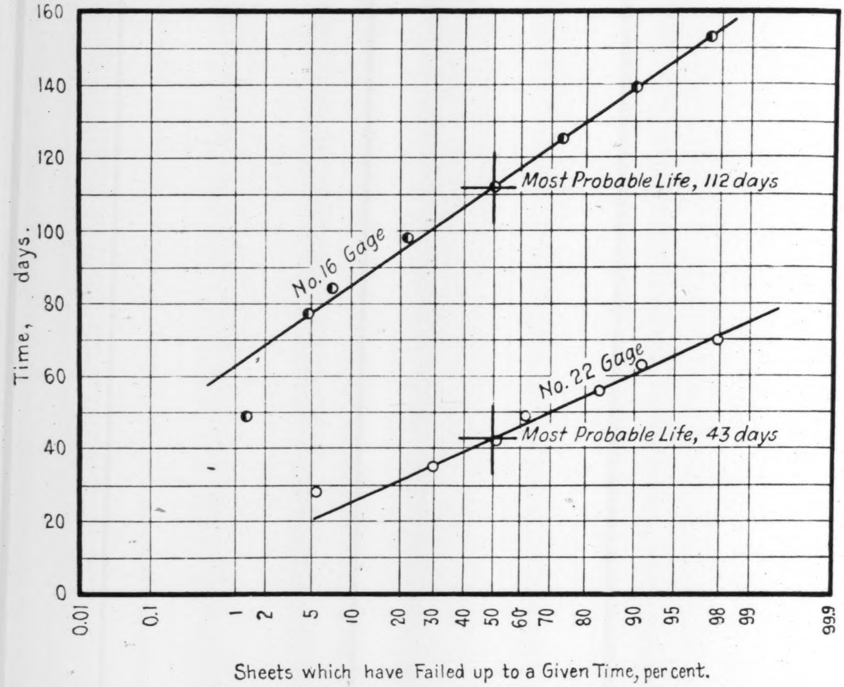

FIG. 3.—Immersion Test Results of All Samples of Iron and Steel Sheets in Calumet Mine Water.

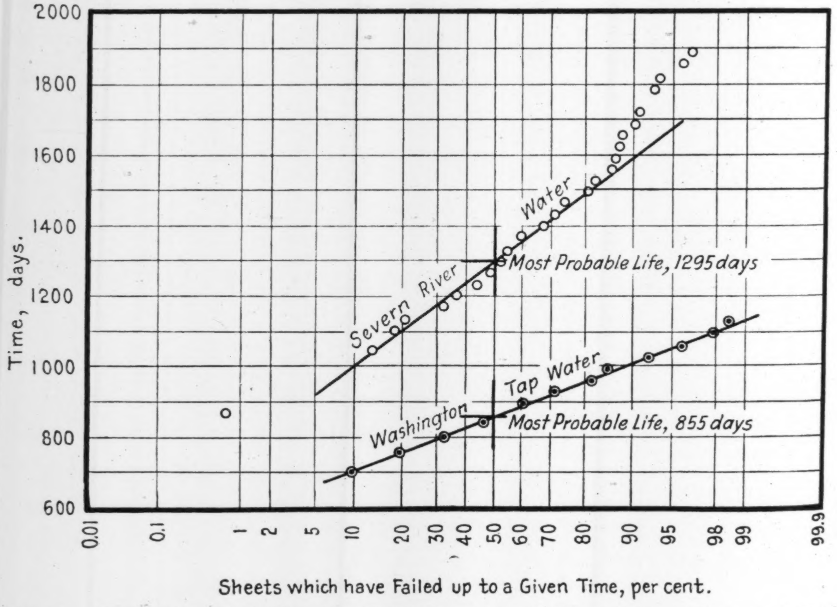

FIG. 4.—Immersion Test Results of All Samples of No. 22 Gage Iron and Steel Sheets in Washington Tap Water and Severn River Water.

Unless there is an extreme difference between two species, there is no possibility of distinguishing them by a test which involves only a small number of samples. Naturally when dealing with a few selected specimens, it is impossible to tell whether they represent the best, the worst, or the average of their type. Even imagining that it were possible to tell which of the specimens fell into the good, average, and bad classes at the beginning, their ratio to each other could not be assumed to correspond to that of the product which they represent.

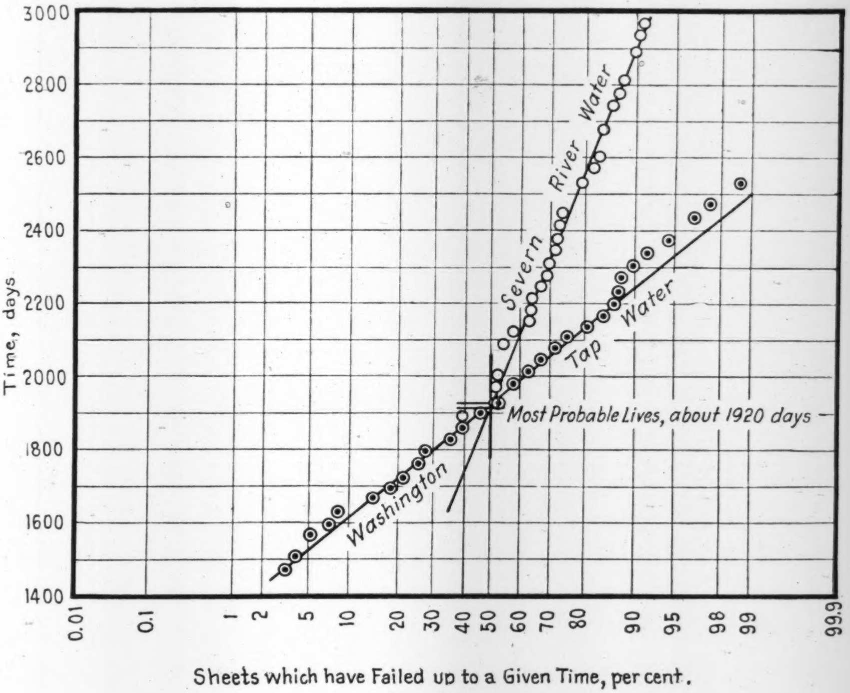

FIG. 5.—Immersion Test Results of All Samples of No. 16 Gage Iron and Steel Sheets in Washington Tap Water and Severn River Water.

APPLICATION TO A.S.T.M. IMMERSION TEST DATA

For illustrative examples, we have chosen the only published results of corrosion experiments in which a number of samples of similar grades have been exposed to the same sets of conditions. No attempt has been made in these studies of the immersion tests to separate the variables which enter the groupings. It has seemed more interesting, and certainly less controversial, to consider those things which pertain to the whole group of samples.

In Figs. 3, 4 and 5 are shown curves which are thought to represent the failures of Nos. 22 and 16 gage iron and steel sheet samples in Calumet Mine water, Washington tap water, and Severn River water. The data from which these curves have been plotted include the results of immersion tests of all samples at the three localities, as reported by Committee A-5 on Corrosion of Iron and Steel.[1] The percentage of failures up to a given time is plotted against the time. In each case, it will be observed that there has been a distribution of failures approximating that required by the probability function. Through the series of plotted values, a straight line has been drawn.

For the reason that the probability function is a symmetrical one, on the summation paper the 50-per-cent mark corresponds to the maximum of the original function (or where the largest number of samples failed), which is the most probable life. Then at the intersection of the line drawn through the experimental points and the 50-per-cent line, the most probable life is shown and is recorded. This is a in Eq. 2.

TABLE I.—MOST PROBABLE LIFE OF SHEET SAMPLES, DAYS.

	NO. 22 GAGE	NO. 16 GAGE
Calumet Mine Water..........................	43	112
Washington Tap Water..........................	855	1920
Severn River Water..........................	1295	1920

An examination of the values in Table I leads one to suspect that the life of No. 16 gage sheets in the Severn River water is in error. The authors are of the opinion that an appropriate explanation of this irregularity lies in the delayed first inspection of the group. All other locations had been cleaned early enough to find the first failures, but in this case, 40 per cent (or 60 samples) were found perforated at the first examination.

It will be observed that the line representing each experiment has a characteristic slope which is a reflection of the range of ages over which failures occur. In Eq. 2 for the probability curve it is h which controls the distribution or spread about the most probable life a. From these facts, it was thought possible to evaluate h from the slope of the line on the summation paper.

By calculation of distribution curves from lines crossing the paper at various angles, it was discovered that:

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[1] For Calumet Mine water test (Fig. 3), Table I, *Proceedings*, Am. Soc. Testing Mats., Vol. 21, pp. 160–163 (1921); for Washington tap water test (Figs. 4 and 5), Table II, Vol. 24, Part I, p. 228 (1924) and Plate IV, Vol. 28, Part I, p. 160 (1928); for Severn River water test (Figs. 4 and 5), Plate III, Vol. 26, Part I, p. 128 (1926) and Plate V, Vol. 28, Part I, p. 160 (1928).

$$h \times \text{slope} = \text{constant} = 0.238 \ldots\ldots\ldots\ldots\ldots (8)$$

when the linear divisions on the paper (measuring approximately 1 cm.) were equal to unity. Having established this relation, h could be calculated for any chosen linear units by inserting a proper factor.

The curves shown for the immersion tests in Washington tap water and Severn River water (Figs. 4 and 5) are drawn with the smallest divisions[1] of the linear vertical scale equivalent to 20 days, or the divisions measuring about 1 cm. equal to 100 days. Hence it is necessary to apply the factor $\frac{1}{100}$ (10^{-2}) in determining the value of h from Eq. 8. Values of h for these cases are shown in Table II.

Knowing a, the most probable life, and h, which is a measure of the distribution about the most probable life, all characteristics of the material or the condition are defined. The criteria of material or a set of conditions are h and a. The best material under a given condition will be the one having the highest value of h and a; h is a measure of uniformity.

TABLE II.—VALUES OF "h."

	MEASURED SLOPE	$h = \left(\frac{0.238}{\text{slope}} \times 10^{-2}\right)$
No. 16 Gage, Severn River Water	2.572	0.00093
No. 16 Gage, Washington Tap Water	0.827	0.00288
No. 22 Gage, Severn River Water	0.775	0.00307
No. 22 Gage, Washington Tap Water	0.397	0.00599

All necessary data regarding service given can be obtained directly from the curve on summation paper. Under some circumstances, however, it may be desirable to calculate distribution curves. We already know h and a and can readily determine k, the fraction which fails at the most probable life. Inspection of Figs. 3, 4, and 5 will show that k is fixed by the slope (or h) if the inspection interval is known. From the constants in the integration of the probability function and calculation, we have found that

$$k = \frac{h}{\sqrt{\pi}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

when the inspection interval is the same as the linear scale value assigned to the division measuring approximately 1 cm. For any other inspection interval, k may be computed by inserting a factor. Of course, h is a constant and its value is fixed by the slope. It is, then, more fundamental than k.

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[1] These divisions on the original drawings were too small for satisfactory reproduction and do not appear in the figures. They are one-fifth of the smallest vertical division in Figs. 3, 4 and 5 as reproduced.—ED.

Having shown that the desirability of a given material for a given set of circumstances is measured by both a and h (or in words, the value of the most probable life and the spread), it might be desirable to bring these to a common unit. This has been done already in every text book on the subject of probability by the use of the probable error. From any table giving the value of the probability integral it can be found that when the probability, P, is equal to 0.5

$$h\,x = 0.4769$$

Substituting r, the probable error, for x

$$hr = 0.4769, \text{ or}$$

$$r = \frac{0.4769}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

The "probable error" has been defined and fixed at a value such that it is an even chance that a random error will be greater or less than its value, from which it follows directly that the value of the probable error can be determined from the intersection of the line drawn through the experimental points with the 25-per-cent and 75-per-cent marks on the paper.

We might go one step further and define the "probable range of failures" as the range between the 25-per-cent and 75-per-cent marks. There is an even chance that any random sample in the class will be within or outside the limits set.

The calculated distribution curves shown in Fig. 2 were computed on the basis of 100-day inspection intervals. A comparison of this figure with Fig. 5 will demonstrate simply the effect of h.

Returning to Figs. 3, 4, and 5, it is to be noted that when all kinds of material are considered as a group, a fairly smooth curve results. From this fact, it may be judged that the spread is characteristic of the service given. It is not possible by this treatment to separate the spread due to variation in material from the spread introduced by the service condition. If a sufficient number of samples of a single kind had been exposed, h for that group would be expected to be similar to the h for the total, but the most probable life, a, might be different. However, it is apparent that the difference in the most probable lives for different kinds of metal is small compared to the spread of a single kind.

It is also to be observed that as the most probable life increases or the condition becomes less severe, the range over which failures are distributed becomes greater.

From these facts, it is evident that where large differences exist in the most probable lives of different materials these may be qualitatively located on a small number of samples but for the general case a few specimens are likely to yield an accidental result. Variation in the service (measured by the most probable life and the spread) can only be gaged from studies in the conditions under which the material serves.

SUMMARY

In summary, the authors believe that in this paper they have shown:

1. That a proper study of the life of ferrous metal must involve an adequate number of samples;

2. That when such numbers are available, results can be obtained by distribution principles when proper classification into groups has been made;

3. That h is a measure of uniformity, either of material or of service condition or a combination of the two. The larger is h, the greater is the uniformity;

4. That the value of a, the most probable life, represents the average service life of the group. The higher is a, the more desirable is the material or the condition. Both a and h must be considered in reckoning desirability, since the combination yielding early failures with a high most probable life is undesirable.

DISCUSSION ON ANALYSIS OF CORROSION DATA

MR. J. H. GIBBONEY.[1]—I have been identified for a number of years with the corrosion work that has been so ably analyzed in these two papers, and I think that the Society can draw a very important lesson from this statistical method of analyzing scientific data. When this corrosion work was planned some fifteen years ago committee procedure with us was not very highly developed, and I am sure that some of the difficulties found in adapting these data to statistical treatment will not be found in future work, due to better planning of all tests by our committee. I do not take exception to the friendly criticism that the authors make regarding the small number of specimens involved in these tests, and the fact that the tests were not inspected at evenly spaced intervals, so that the data might lend themselves more truly to statistical treatment. When the tests were planned years ago we simply did not at that time appreciate the significance of such details.

I wish at this time to thank the authors for presenting these splendid papers. I am sure that if other committees will take the time to study the methods that have been followed in these papers, greater use can be made of the statistical treatment of scientific data in presenting to better advantage the results of investigations to the Society.

MR. H. L. H. SMITH.[2]—I think that it is an extremely interesting event in the Society's proceedings to have the method of statistical analysis used in the analysis of test work. I assume that it is relatively a new thing. I was a member of the Society for a good many years and then I dropped out and only recently re-entered, so I am not sure whether the papers that have been read this morning are as new to the Society as they seemed, off hand, to me. The work of statistical analysis is of course relatively new, anyway. Fifteen years ago only the masters of it who were establishing it had any acquaintance with it at all, and more and more in engineering work, it is entering ordinary procedure and analysis. I have not had an opportunity to read the papers as a whole, but just looking over them as they were abstracted, I wondered whether too much emphasis was laid on the comparison of the actual results with the normal curve, and whether something useful might result if the skew curve were used in analysis. At any rate, the whole movement is of extreme interest, and I think we will all see it used to a larger and larger extent as the years go on.

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[1] Chief Chemist, Norfolk and Western Railway Co., Roanoke, Va.

[2] Research Engineer, New England Power Construction Co., Boston, Mass.

MR. ANSON HAYES[1] (*authors' closure by letter*).—Mr. Gibboney's endorsement of the statistical method of analysis of corrosion data is indeed appreciated. As a means of definitely establishing the numbers of structures required in order to get significant results, the authors wish to emphasize the importance of making statistical studies of structures already in service. This may be applied to culvert service, roofing service, siding service, as well as buried pipe service.

Where various competing materials are sufficiently interspersed throughout a given region, it seems very likely that more definite information in regard to the relative merits of competing material can be obtained than would be possible in any specially designed experimental test. This is true because of the more satisfactory nature of the data as the number of structures involved in a survey increases. In many cases, it would not be economically feasible to put up a test which would involve an adequate number of samples.

In regard to the remarks made by Mr. H. L. H. Smith, no doubt it might be possible to fit corrosion data from a rather broad range of service conditions into some skew curve. However, in the data investigated thus far in fields other than those represented in this paper, in nearly all cases it is possible to select a range of service conditions which fit the data into one or the other of two symmetrical distribution curves. One of them is known as the arithmetic probability curve, and the other is known as the logarithmic probability curve.

We are quite in agreement with Mr. Smith that the possibilities of the statistical method have not been taken advantage of in researches which have generally been carried out in production plants. Here where a process is made up of a large number of steps, each of which has variables involved in its operation which cannot be controlled independently, a statistical method offers about the only possibility of getting accurate information in regard to the influences of a given variable on the finished product. The use of the method will certainly be greatly extended in the very near future.

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[1] Chief Chemist, The American Rolling Mill Co., Middletown, Ohio.

METHOD OF MAKING COPPER REPLICAS OF CORRODED METAL OBJECTS

BY E. S. TAYLERSON[1]

SYNOPSIS

As a part of the corrosion work of Committee A-5 on Corrosion of Iron and Steel, a series of riveted plates has been exposed to the influence of sea water. To obtain an accurate record of the progressive corrosion of the rivets and the portions of the plates surrounding them, copper replicas in relief are to be made at regular intervals.

A detailed description of the method employed is given, including the making of wax molds either by casting or by means of pressure, with subsequent deposition of copper as in the regular electrotyping process.

The application of the method is largely limited to flat objects, where pitting is not excessively deep, and is of use only where the samples are thoroughly cleaned from corrosion products at certain intervals during the test.

Committee A-5 on Corrosion of Iron and Steel of this Society has been making a series of tests on rivets of different composition in riveted structures exposed to sea water. Considerable trouble has been encountered with the failure of riveted joints, and the tests were planned to determine the effect of sea-water immersion where steel rivets and plates vary considerably in composition.

In March, 1928, Sub-Committee V, of Committee A-5, on Total Immersion Tests under whose direction this work is being undertaken, inspected the riveted plates immersed in the sea at Key West, Fla., and found that, due to the heavy marine growth, it was necessary to clean the plates thoroughly at each inspection if significant observations were to be made. It was found that field measurements, taken in an attempt to determine the progressive corrosion of the rivets, were inadequate due to the slow rate of corrosion and the irregular surface of the rivets. The committee at that time discussed briefly possible means of recording the condition of the rivets at each inspection by some means of taking impressions of each rivet. It was also decided to sand blast the plates lightly for the removal of coral and rust prior to each inspection. Photographs were taken of the plates, but they did not show sufficient detail to be entirely satisfactory.

This subject was further discussed at a subsequent meeting of the committee, with the result that at a meeting in June Mr. E. L.

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[1] Engineer of Tests, American Sheet and Tin Plate Co., Pittsburgh, Pa.

Chappell exhibited a plaster replica, which he had made from a corroded plate furnished by the Navy Department. It was suggested at that time that the difficulties of employing plaster in the field might be too great, and that possibly a wax mold would be more suitable. A supply of ozocerite, known to the trade as standard electro-wax was obtained, and an experimental mold was made by casting. When this mold was being electrotyped, the electrotyping company was supplied with a corroded sheet, an impression of which was made in the wax by hydraulic pressure. The electrotype from this mold showed so much better detail than that from the cast mold that arrangements were made to copy some of the Key West riveted plates by this process at the next inspection.

Due to the effective cooperation of the personnel of the Naval submarine base at Key West, the committee now has an accurate record of the condition of a number of the corroded rivets.

The following description of the method employed is given as applicable in making permanent records of the condition of corroded metal objects, where the objects can be thoroughly cleaned at intervals during the test.

METHOD OF PROCEDURE

Regular electrotyper's wax is melted, taking care to avoid overheating, and poured out in a thin film on a metal backing plate, preferably of type metal, copper, or brass. To make sure of the adherence of the wax layer to this plate, it is desirable that the plate should be rough. The surface of the molten wax should be freed from air bubbles by drawing a card over it, and after cooling, should be smoothed by means of a sharp scraper or knife, though this can be done at the electrotyping plant to better advantage, as special equipment is available.

The thickness of the wax should be adjusted to suit the depth of the impression desired, though it is better not to exceed $\frac{1}{4}$ in., as otherwise the wax will ooze out excessively at the edges of the mold when pressure is applied. Thicker layers, however, are necessary if deep impressions are to be made.

The wax plates should be carefully inspected for surface defects and any such may be eliminated by passing a flame rapidly over the surface.

The article to be copied must be thoroughly freed from corrosion products by some method which does not injure the underlying metal surface. The procedure used in the case of the riveted plates was to sand blast them lightly. While sand blasting produces a slightly matte surface, it does not seem to remove an appreciable amount of metal.

The surface of both the metal object and the wax mold are thoroughly covered with a thin layer of electrotyper's graphite, applied dry with a camels hair brush. The object of the graphite is to prevent adherence of the wax to the metal surface. The graphite must be free from grit. Excess graphite is removed from the interstices of the corroded surface by vigorous brushing and blowing. If these precautions are not taken, the final copper reproduction will be lacking in detail.

The metal object and the wax should then be brought to a temperature of approximately 110° F., so that the wax will flow more freely when pressure is applied. This pressure may be provided by any suitable means, depending upon the area to be molded. In general, for small objects a heavy vise or C-clamp is suitable, while for larger surfaces, a screw or hydraulic jack with suitable framework, or the compression plates of a universal testing machine, would be satisfactory. As uniform distribution of pressure is important, it is advisable to place a thick pad of paper between the press platen and the metal object. The pressure should be applied gradually, leaving ample time for entrapped air to escape and the wax to flow into every depression.

In the case of the riveted plates, pressure was applied over an area of approximately one square foot by means of a hydraulic mandrel press. In the case of larger areas, it is better to use a heavy steel or cast-iron plate, designed to transfer the pressure from the head of the ram of the press to the flat surface surrounding the rivets.

The wax at the edges of the mold will exude slightly, and this part of the mold should be discarded, as it will be severely distorted. Upon removal of the pressure, the mold is readily separated from the original, and it should be carefully examined to see that the wax has penetrated to the bottom of every depression. With a little care the mold can be replaced in its original position in case further pressure is necessary.

The pressure method of making the mold is not always the most suitable. If a cast mold is desired, the wax should be heated to a temperature just above the melting point and then poured over the warm object, provision having been made for a retaining frame, which may be of wood or clay or other suitable material. Care should be taken to detach any air bubbles from the graphite-coated surface of the metal during the process of pouring the mold, otherwise such bubbles will cause protuberances on the finished reproduction.

Other methods of molding, such as those employing gutta percha and compositions of various kinds, are described in the literature. In some cases even sheet lead can be used where sufficient pressure is available and the impressions desired are extremely shallow. These latter methods, however, have not proved practicable for the investigation described in this paper.

The graphite coating entirely prevents the adherence of the wax to the metal object; but if the object is to be further exposed to corrosion, it is essential that all traces of graphite be removed. This is usually accomplished by the use of gasoline and a stiff brush, though it is necessary to use a second application of clean gasoline to remove the last traces of graphite. After the final cleansing, a white cloth should be rubbed over the surface and inspected to make sure that all graphite has been removed.

If it is necessary to ship the molds, they should be so secured that the surface will be protected from abrasion. The wax molds are very brittle at low temperatures, and it is undesirable to ship them during the winter months, unless they are well protected from shock.

The molds are then trimmed, properly identified, thoroughly coated with graphite by the wet process, metallized in the usual manner by the use of copper sulfate solution and fine iron filings, and then suspended in a copper sulfate bath for the deposition of the copper coating by electrolysis. To avoid risk of distortion, it is advisable to deposit a heavy copper "shell," if accurate measurements of the depth of pitting are to be made. When the copper has been deposited, the wax mold is warmed and detached and the adherent graphite removed from the copper replica by means of kerosine or gasoline. The copper shell should be reinforced by a thin layer of solder over the entire back surface, using zinc chloride as a flux.

Acknowledgment.—The assistance of Mr. E. G. J. Gratz, President of the International Association of Electrotypers, whose knowledge of the art and experience in the field of electrotyping were invaluable, is gratefully acknowledged.

THE INFLUENCE OF CORROSION ACCELERATORS AND INHIBITORS ON FATIGUE OF FERROUS METALS

By F. N. Speller,[1] I. B. McCorkle[1] and P. F. Mumma[1]

Synopsis

The data reported in this paper follow up that given last year in a paper under the same title by the authors in which the marked influence of inhibitors in preventing corrosion-fatigue was demonstrated. In the present paper the influence of a wide range of chloride concentrations on corrosion-fatigue of a similar standard steel is reported, together with the influence of various amounts of inhibitors in retarding corrosion-fatigue with the same range of chloride concentrations. The amount of inhibitor required increases considerably with the chloride concentrations.

The effect of localized corrosion in reducing the endurance limit under stress-corrosion, with or without inhibitors, is also noted. As in ordinary corrosion the stability of surface films is evidently a factor of prime importance in corrosion-fatigue. Stress and localized corrosion tend to damage these protective films and greatly reduce the life of the metal.

Introduction

In our paper on this subject last year,[2] it was shown that with water similar to Pittsburgh tap water:

1. Corrosion under stress could be stopped with inhibitors (for example, sodium dichromate), and that under these conditions the damage due to corrosion, combined with fatigue, was absent. In other words, the corrosion-fatigue limit corresponded to or slightly exceeded the air fatigue limit.

2. When "local" corrosion was present, the efficiency of the inhibitor in overcoming corrosion-fatigue was greatly reduced.

The present paper is based on a study of variations in the endurance of a similar steel (containing 0.42 per cent of carbon) when corroded by water with sodium-chloride (NaCl) concentrations varying between 25 and 30,000 parts per million.

Tests were made of the effect of various amounts of inhibitor on the number of stress cycles with a fixed stress slightly below the air endurance limit, for the salt concentration range stated above (Fig. 2). The relative inhibitive value of sodium dichromate ($Na_2Cr_2O_7$) and sodium chromate (Na_2CrO_4) is compared in this series. The normal chromate has been found to be much more efficient than the dichromate under stressless corrosion.

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[1] Department of Metallurgy and Research, National Tube Co., Pittsburgh, Pa.

[2] F. N. Speller, I. B. McCorkle and P. F. Mumma, "Influence of Corrosion Accelerators and Inhibitors on Fatigue of Ferrous Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 159 (1928).

The passive films formed after 98 hours in solutions of $Na_2Cr_2O_7$, which show some temporary protection in running water, apparently do not increase the endurance under stress and corrosion combined. McAdam[1] has already shown that films formed by anodic corrosion of light aluminum alloys are ineffective in protecting the metal against corrosion-fatigue.

The maintenance of a protective film seems to be essential to protection against corrosion-fatigue as well as against stressless corrosion. External conditions which set up even a slight difference in potential have a marked influence in destroying these films, especially when simultaneously the metal is subjected to cyclic stress and the surface is bombarded by chlorine ions. So the several factors tending to cause local corrosion, and the speed with which breaks in the surface film are repaired, seem to be the main opposing forces at work in corrosion-fatigue as in ordinary stressless corrosion.

TABLE I.—COMPOSITION AND MECHANICAL PROPERTIES OF MATERIAL.

	Carbon, per cent	Manganese, per cent	Phosphorus, per cent	Sulfur, per cent	Silicon, per cent	Nickel, per cent	Chromium, per cent	Yield Point, lb. per sq. in.	Tensile Strength, lb. per sq. in.	Elongation in 2 in., per cent	Reduction of Area, per cent
Material A.....	0.42	1.00	0.014	0.034	0.085	.....	.....	52 090	95 920	24.6	54.3
Material B.....	0.074	0.43	0.037	0.020	0.731	None	18.56	51 240	76 470	31.0	72.0

It is thought that rust-resisting steels owe their higher resistance to the stability of the passive film formed. Probably it takes a definite period of time to rebuild these films when once broken, as in the case of passive films formed by some film-building compound in solution. Figure 4 gives some corrosion-fatigue tests on iron containing 18.5 per cent of chromium and 0.07 per cent carbon, in air (that is, with no corrosion), with general corrosion and where local corrosion is introduced by placing a rubber ring around the center of the specimen. A definite though relatively slight drop in the endurance limit under local corrosion occurs, apparently due to the small disintegrating potential imposed on the surface under corrosion in addition to that caused by the stress. It is an interesting question, how much of the local corrosion under stress is due to the stress itself and how much to other influences causing differences in potential on the surface of the metal.

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[1] D. J. McAdam, Jr.. "Fatigue and Corrosion-Fatigue of Metals," *Proceedings*, Internat. Congress for Testing Mats., September, 1927, p. 337.

As the resistance of stainless iron to corrosion is so decidedly changed by a slight difference in local potential, both under stressless and cyclic stress corrosion, it now seems all the more certain that such metals owe their extraordinary resistance to corrosion to the presence of a passive film rather than to an inherent low solution pressure. Thus, corrosion-fatigue testing affords another useful means of studying metal protective films which are often invisible.

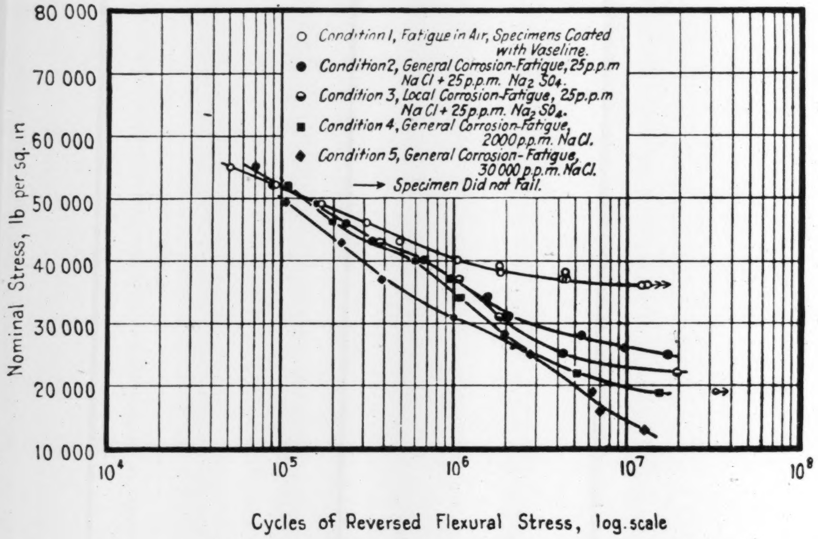

FIG. 1.—Fatigue in Air and Corrosion-Fatigue of Material A.

MATERIAL AND TEST METHODS

Two materials were used: material A, a basic open-hearth steel in the form of seamless tubing, size 7 in. outside diameter by 0.571-in. wall; material B, commercial stainless iron, $\frac{5}{8}$-in. diameter bar. Both materials were tested in the as-rolled condition and all test specimens were machined from a single tube or bar. Table I gives the chemical composition and the mechanical properties of material.

The tests were made on the R. R. Moore rotating-beam fatigue testing machine which has been described elsewhere.[1] The special test specimen used and the manner of circulating the waters used in testing have been described in the previous paper.[1] For both ordinary fatigue and corrosion-fatigue the speed of testing was 1790 r.p.m.

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[1] R. R. Moore, "Effect of Corrosion upon the Fatigue Resistance of Thin Duralumin," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 128 (1927).

RESULTS OF TESTS ON MATERIAL A

Fatigue in Air and General Corrosion-Fatigue in Various NaCl Solutions:

Figure 1 presents curves of the stress-cycle type plotted to the usual semi-logarithmic scale and superimposed to aid inspection. The following conditions of testing are represented:

1. Fatigue in air, specimen coated with vaseline.
2. General corrosion-fatigue in synthetic Pittsburgh tap water (25 p.p.m. NaCl plus 25 p.p.m. Na_2SO_4 in distilled water).
3. Local corrosion-fatigue, using a rubber washer ($\frac{7}{16}$ in. in diameter by $\frac{1}{16}$ in thick) snugly fitted around the middle of the test specimen using the same water as in the case of general corrosion-fatigue above.
4. General corrosion-fatigue in a solution of 2000 p.p.m. NaCl in distilled water. This is about the maximum amount of salt that has been found in oil-well mud fluids.
5. General corrosion-fatigue in a solution of 30,000 p.p.m. NaCl in distilled water, which approximates sea water.

Figure 1 needs no particular comment. The results are quite regular and the endurance limit on a 10,000,000-cycle basis is 36,000 lb. per sq. in. It will be noted that the corrosion-fatigue curves show considerable slope beyond 10,000,000 cycles. Therefore, comparison with the air curve is made for the stress at which a curve cuts the abscissa for 10,000,000 cycles. On this basis the damage of corrosion-fatigue may be computed as follows:

CONDITION	STRESS FOR 10 000 000 CYCLES, LB. PER SQ. IN.	DAMAGE TO AIR FATIGUE AT 10 000 000 CYCLES, PER CENT
2	26 000	27.8
4	20 000	44.4
5	14 000	61.1

The progressive lowering of the fatigue values as the concentration of NaCl is increased is thus definitely shown. The rate of damage rapidly decreased, but it is important to note that the total damage continues to increase.

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[1] F. N. Speller, I. B. McCorkle and P. F. Mumma, "Influence of Corrosion Accelerators and Inhibitors on Fatigue of Ferrous Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 159 (1928).

There does not seem to be any connection between the amount of local or total corrosion without stress and the amount of damage by corrosion-fatigue, nor would this be expected. For example, McAdam has shown that corrosion-fatigue is less in ordinary tap water than in brackish water of the Severn River at Annapolis (1.55 per cent salt), whereas the total immersion tests conducted by the Society's Committee A-5 on Corrosion of Iron and Steel[1] indicate the opposite for stressless corrosion, judging by the rate of perforation of sheets in the Severn River water and Washington city water. The reason for this apparently is that the Severn River water forms a protective but brittle scale, and that this scale is easily cracked under stress, permitting attack of chlorine ions. Ordinarily, corrosion in pure NaCl reaches a maximum at about 5000 p.p.m., from which concentration the rate slowly decreases as the oxygen solubility decreases.[2]

Local Corrosion-Fatigue:

This refers to condition 3 above. The damage to the air fatigue value at 10,000,000 cycles was 36.1 per cent, which is 8.3 per cent greater than the damage due to general corrosion-fatigue for the same water. The curve shows that the damaging effect of local corrosion in this case did not operate until the stress was dropped below the air-endurance limit.

General and Local Corrosion-Fatigue in Various NaCl Solutions With Inhibitor Additions:

It became important to determine how useful various common inhibitors would be in lessening the damage of general and local corrosion-fatigue for the various salt solutions. For this work it was expedient to adopt a definite stress for all the tests and to vary the amount of inhibitor added to the base solutions for the individual tests. The results are in the form of curves, plotting the concentration of inhibitor against the number of cycles at the given stress. The definite stress of 34,000 lb. per sq. in., 2000 lb. per sq. in. below the air-endurance limit for the material, was chosen for three reasons: The performance of specimens stressed *at* a determined air-endurance limit is often erratic and this stress is therefore useless for comparative tests; all thermal effects are eliminated and the action is purely corrosion-fatigue; and a stress range close to the determined endurance limit must be used if the test results are to have practical value.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 160 (1928).

[2] F. N. Speller, "Corrosion—Causes and Prevention," McGraw-Hill Book Co., New York, p. 173.

A description of the conditions for the various test series is as follows:

6. General corrosion-fatigue in synthetic Pittsburgh tap water, 25 p.p.m. NaCl plus 25 p.p.m. Na_2SO_4, plus various amounts of $Na_2Cr_2O_7$.
7. General corrosion-fatigue, 2000 p.p.m. NaCl, plus various amounts of $Na_2Cr_2O_7$.
8. General corrosion-fatigue, 30,000 p.p.m. NaCl, plus various amounts of $Na_2Cr_2O_7$.
8. (*a*) General corrosion-fatigue, 30,000 p.p.m. NaCl, plus 2500 p.p.m. $Na_2Cr_2O_7$.
9. Local corrosion-fatigue, 2000 p.p.m. NaCl, plus various amounts of $Na_2Cr_2O_7$. A rubber washer ($\frac{7}{16}$ in. in diameter by $\frac{1}{16}$ in. thick) was fitted snugly around the middle of the test specimen.
10. General corrosion-fatigue, 2000 p.p.m. NaCl, plus various amounts of Na_2CrO_4.
11. General corrosion-fatigue, 30,000 p.p.m. NaCl, plus various amounts of Na_2CrO_4.
12. Local corrosion-fatigue, 2000 p.p.m. NaCl, plus various amounts of Na_2CrO_4. A rubber washer ($\frac{7}{16}$ in. in diameter by $\frac{1}{16}$ in. thick) was fitted snugly around the middle of the test specimen.

All the solutions were made up with distilled water.

Except for condition 8 (*a*), which will be referred to later, the graphs of the results are grouped in Fig. 2. Cartesian coordinates have been used as this type of plotting gives a better picture of relative values. A summary may be drawn from these data, keeping in mind that the statements hold fully only for the steel used for the tests:

With mild salt solutions, of the order of 25 p.p.m. NaCl, general corrosion-fatigue was overcome with the addition of 175 p.p.m. $Na_2Cr_2O_7$ (condition 6).

With a solution carrying 2000 p.p.m. NaCl, general corrosion-fatigue was overcome with the addition of 2500 p.p.m. Na_2CrO_4 (condition 10). Progressive additions of $Na_2Cr_2O_7$ failed to prevent corrosion-fatigue; at 16,000 p.p.m. specimen life was 2.2×10^7 cycles but the curve showed a slope indicating that further additions would be beneficial (condition 7). Amounts of $Na_2Cr_2O_7$ below about 2000 p.p.m. gave better results than corresponding amounts of Na_2CrO_4.

With a base solution of 30,000 p.p.m. NaCl, corrosion-fatigue was inhibited at approximately 8000 p.p.m. Na_2CrO_4 (condition 11), *when* *conditions of strictly general corrosion prevailed throughout the course of a test.* That is, for about 8000 p.p.m. or above the normal effect in corrosion-fatigue is protective, but Fig. 2 indicates that this protection may be broken down by unknown factors that introduce local corrosion. Thus the procedure for the two tests at 8000 p.p.m. (condition 11) was the same, yet one specimen ran millions of cycles while the life of the other was most probably cut short by the early and accidental introduction of a condition of local corrosion. The failure of the specimen tested at 12,000 p.p.m. is another illustration. (To aid identity, a circle has been drawn around individual scattered points for condition 11). When $Na_2Cr_2O_7$ was used corrosion-fatigue could not be prevented (condition 8). Amounts over 8000 p.p.m. were not beneficial and the curve became vertical at slightly over 6,000,000 cycles. Amounts of $Na_2Cr_2O_7$ below about 6000 p.p.m. were more effective than corresponding amounts of Na_2CrO_4.

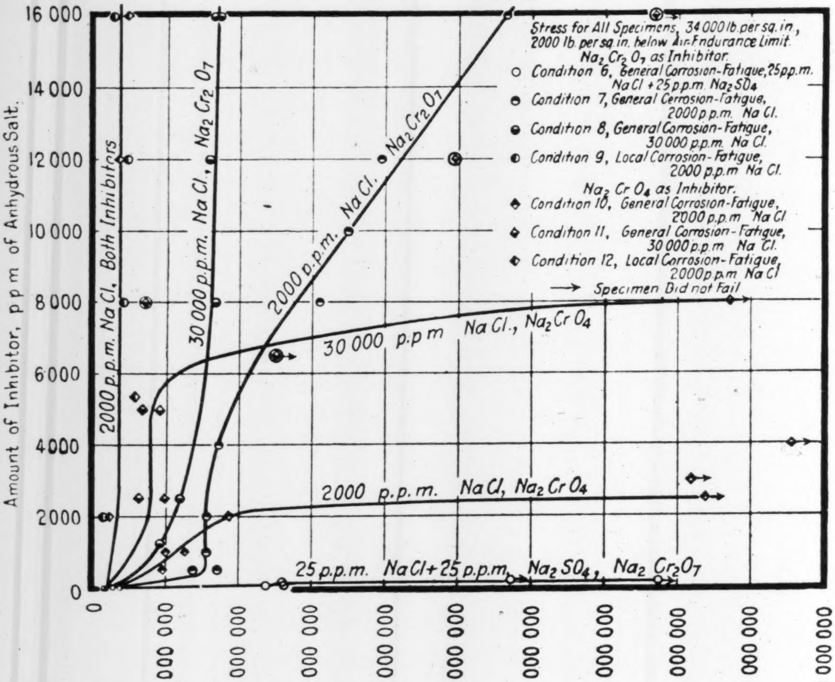

FIG. 2.—Effect of Inhibitor Concentration on Corrosion-Fatigue of Material A.

Using a base solution of 2000 p.p.m. NaCl and a rubber washer at the middle of the specimen, the results showed that Na_2CrO_4

(which effectively prevented general corrosion-fatigue for the same base solution) and $Na_2Cr_2O_7$ were equally ineffective in overcoming local corrosion-fatigue (conditions 12 and 9). Only one curve has been drawn in Fig. 2 to represent the performance of both inhibitors with local corrosion because the results were so nearly similar. This curve becomes vertical at an approximate concentration of 4000 p.p.m. of either inhibitor, corresponding to a specimen life of about 1.4×10^6 cycles. For the same condition of local corrosion, but with no inhibitor in the water (that is, zero ordinate for the curve) specimen life was 7.73×10^5 cycles. The use of inhibitors, therefore, extended the life of these specimens only 1.8 times.

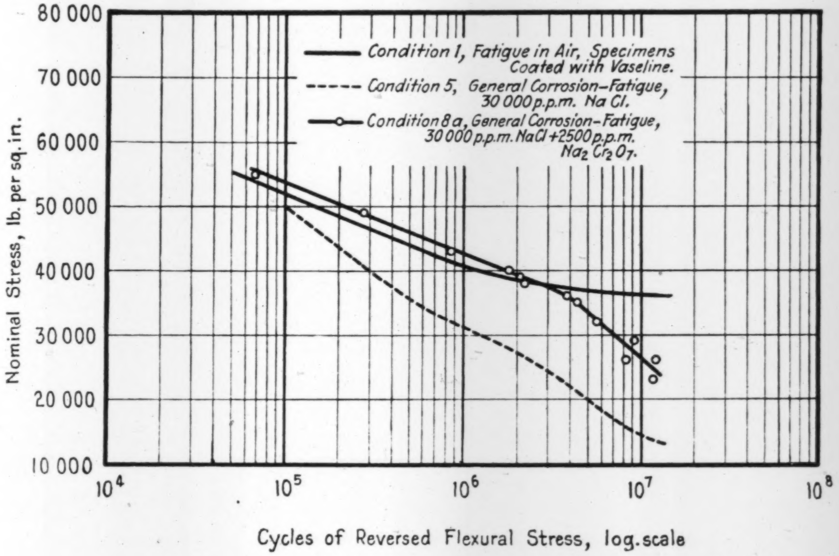

FIG. 3.—Effect of Inhibitor on Corrosion-Fatigue of Material A.

For comparison with the above results on inhibition of corrosion-fatigue, the following brief summary is given of the relative values of $Na_2Cr_2O_7$ and Na_2CrO_4, in preventing corrosion under static conditions:

In distilled water, corrosion is prevented with a smaller addition of Na_2CrO_4 than with $Na_2Cr_2O_7$.

In natural waters, complete protection is reached at a concentration under 250 p.p.m. Na_2CrO_4 and no change may be expected by further additions. In fact, further increase may be less effective (depending upon the character and quantity of salts in solution) and may result in some local attack.

In strong NaCl solutions, increments of Na_2CrO_4 are progressively beneficial. In the same solutions up to a concentration of 250 p.p.m., $Na_2Cr_2O_7$ is also progressively beneficial, but further increase under these conditions seems to be detrimental.[1]

Additional Illustration of the Effect of Inhibitors:

For condition 8 in Fig. 2, it was shown that in the "corrosion-fatigue" range of stress, where 34,000 lb. per sq. in. is near the upper limit of corrosion-fatigue, it is not possible to prevent damage through additions of $Na_2Cr_2O_7$. In Fig. 3, the graph for condition 8 (*a*) is an extension of the stress range, the usual stress-cycle curve. The graphs for condition 1 and condition 5 in Fig. 1 have also been drawn in Fig. 3 for comparison. It may now be seen that when the stress range was extended above the air-endurance limit, 2500 p.p.m. $Na_2Cr_2O_7$ added to the 30,000 p.p.m. NaCl solution gave results in general corrosion-fatigue which were superior to the results for tests in air. The increased performance above the air-endurance limit was realized in spite of the general corrosion of the specimens. For example, the test specimens for 40,000 and 39,000 lb. per sq. in. stress were covered with a fine, powdery, fairly adherent, dark-brown mantel of corrosion products. Longitudinal micrographs of the specimens

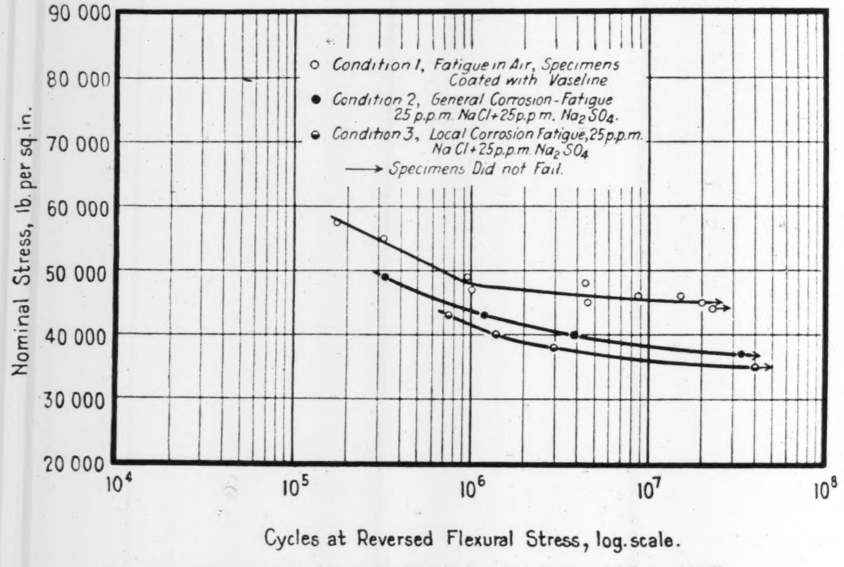

FIG. 4.—Fatigue in Air and Corrosion-Fatigue of Material B.

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[1] J. N. Friend and J. H. Brown, "The Action of Aqueous Solutions of Single and Mixed Electrolytes upon Iron," *Journal*, Iron and Steel Inst., Vol. 83, p. 125 (1911).

showed myriads of small pits along the surface. One explanation is that for the conditions given, corrosion pitting was not intense enough at centers to counteract the cooling effect of the medium. As the air-endurance limit was approached the thermal effect was eliminated and the presence of pits caused the curve to drop below the air curve, at a fairly constant rate of damage. Whether anything like a corrosion-fatigue limit can be approached has not yet been determined.

Another explanation of this peculiar effect may be a decrease in the film-building power of the inhibitor as the pits at the most strained part become deeper, due to the increase in potential between the stressed and less stressed parts. The dip in curve with inhibitor probably corresponds with the point where the protective film cannot be reformed under cyclic stress and rapid pitting begins.

RESULTS OF TESTS OF MATERIAL B

The stainless iron was tested under three conditions:

1. Fatigue in air, specimens coated with vaseline.
2. General corrosion-fatigue in synthetic Pittsburgh tap water, 25 p.p.m. NaCl plus 25 p.p.m. Na_2SO_4 in distilled water.
3. Local corrosion-fatigue, using a rubber washer ($\frac{7}{16}$ in. in diameter by $\frac{1}{16}$ in. thick) snugly fitted around the middle of the test specimen and the same water as in the case of general corrosion-fatigue above.

Curves of the results are shown in Fig. 4. They are of the stress-cycle type and are superimposed to aid comparison. For condition 1, the endurance limit on a 10,000,000 cycles basis is taken as 45,000 lb. per sq. in. The damaging effect of general and local corrosion fatigue is computed to this basis:

CONDITION	STRESS FOR 10 000 000 CYCLES, LB. PER SQ. IN.	DAMAGE TO AIR FATIGUE AT 10 000 000 CYCLES, PER CENT
2	38 000	15.6
3	36 000	20.0

For this material, the damaging effect of corrosion-fatigue is not so great as for material A, an observation which is consistent with its superior performance under ordinary corrosion. The graph for the condition of local corrosion-fatigue shows a definite, but slight further lowering of values, indicated to be about 4.4 per cent in the region of 10,000,000 cycles.

In studying any phase of the corrosion phenomenon, the importance of the stability of corrosion films soon becomes apparent and it is interesting to compare the effect of this factor in ordinary corrosion and with the addition of stress.

It is now generally agreed that under ordinary corrosion, for iron or steel at least, the environment controls the rate rather than any difference in the metals themselves. There is an abundance of proof of this principle. This is mainly due to the different effect of corroding media on the physical properties of the surface films. For instance, copper steel forms a protective skin in air, but this is lacking, as with ordinary steel, in water; hence, the difference in life of copper steel in air and water is not surprising. Such films form on iron and other metals in air, even at ordinary temperature. In water, corrosion-retarding films form at the cathode and anode. Most of these films are oxides and are cathodic to the metal. Passivity has been proved by Evans to be due to formation of a surface film, usually too thin to be visible. These films are easily injured and where the metal is exposed in a conducting solution, they are cathodic to the metal. The anodic area being relatively small, the pitting may be expected to be proportionately deep under these conditions. These facts have been well established with respect to stressless corrosion.[1] In corrosion-fatigue under running water, other factors, namely, stress and cooling effect, are introduced. The latter tends to raise the curve above the air curve; the former acts mainly to break the surface film, which is quickly rebuilt at first, but more slowly as the pit becomes deeper. Very thin stable films, such as those formed by chromates in pure water, are flexible and may resist a stress equal to the endurance limit of the metal in air for some time. Thicker and less resistant films break more easily under stress.

Rust-resisting irons owe their resistance to protective films formed in air. These metals pit when exposed to slight differences in potential as in soil or in contact with other material in water. The same factors, as we have shown, cause a reduction in endurance limit of stainless iron under corrosion-fatigue. Later study of corrosion-fatigue has convinced us that this is strictly a corrosion problem controlled by the same factors as ordinary corrosion, up to the point where pitting gains the upper hand; after this it is ordinary fatigue on an area progressively reduced by intensely localized corrosion. While there is some evidence that strain makes a metal anodic to unstrained parts, we believe that this plays a relatively small part in corrosion-fatigue.

It will be useful to investigate the influence of stress *per se* and in combination with other corrosion factors that are now better understood.

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[1] F. N. Speller, "Corrosion—A Problem in Protective Coatings," *Industrial and Engineering Chemistry*, Vol. 21, p. 506 (1929).

CONCLUSIONS

From the foregoing data, we draw the following deductions:

1. Under corrosion-fatigue, the endurance or specimen life decreases as the chloride contents increase.

2. In water with low saline contents (25 p. p. m. NaCl and 25 p. p. m. Na_2SO_4) even a slight external influence causing localized corrosion reduces the corrosion-fatigue endurance limit decidedly either with or without inhibitors in the corroding solution. This applies to the stainless variety of steels as well as to ordinary steels without inhibitors.

3. The *strength* of inhibitor required to prevent failure under corrosion-fatigue (as with ordinary stressless corrosion) increases considerably with chloride concentrations. Within the range in which these inhibitors are effective, after about five million cycles the normal chromate gives better results than the dichromate.

4. The evidence points to the stability of the surface film as being a factor of first importance in corrosion-fatigue as in ordinary corrosion, and indicates that the factors which cause localized corrosion without stress also play a very prominent, if not a controlling, part in corrosion-fatigue.

5. Stress in the phenomenon of corrosion-fatigue is essential in that it breaks the surface film and brings into play electrolytic factors tending to cause concentrated corrosion at the most strained part, but stress by itself probably plays only a minor part directly in accelerating corrosion.

Acknowledgment.—The valuable assistance rendered by Mr. H. M. Bridgham of our Research Laboratory in analyzing data in the preparation of this paper, is hereby acknowledged.

[For Discussion on Corrosion-Fatigue of Metals, see page 304.—ED.]

CORROSION OF METALS UNDER CYCLIC STRESS[1]

By D. J. McAdam, Jr.[2]

Synopsis

Part I gives a brief résumé of previous work, an outline of the continued investigation, and a description of material and methods.

Parts II to VII, inclusive, discuss the effect of stress, time and number of cycles on corrosion. The resultant fatigue limit is used as a criterion of the effect of corrosion. The interrelationship between these four variables has been illustrated by various types of two-dimensional diagrams and by a three-dimensional diagram. The metals investigated are ordinary steels, duralumin, monel metal and stainless iron.

"Damage" as used in the paper means the lowering of the fatigue limit due to corrosion with or without cyclic stress. The effect of cyclic stress is first illustrated by variable-damage diagrams, Type 5. For ordinary steels and duralumin, Type 5 graphs are "retarded-damage" graphs. For monel metal and stainless iron, Type 5 graphs are "accelerated-damage" graphs. From these Type 5 graphs, "constant-damage" graphs of various types are developed.

The net effect of cyclic stress on corrosion is illustrated by constant-net-damage diagrams. By "net-damage" is meant damage in excess of the amount that would be caused in the same time by stressless corrosion. The net-damage diagrams indicate that for these metals, and possibly for all metals, the stress-time-cycle relationship for constant net-damage is an exponential relationship. This relationship for all ordinary steels, in fresh or salt water, can be represented by practically the same diagram. For ordinary steels, and probably for duralumin, any stress cycle, however small the stress range, accelerates corrosion. For these metals, and probably for other easily corrodible metals, there is apparently no limiting stress below which the effect of cyclic stress on corrosion decreases abruptly. For monel metal and stainless iron it has not yet been determined whether or not there is such a limiting stress.

PART I.—OUTLINE OF INVESTIGATION, DESCRIPTION OF MATERIAL AND METHOD

Previous Investigation of Corrosion-Fatigue at Naval Engineering Experiment Station

Results of a general survey of corrosion-fatigue phenomena at the Naval Engineering Experiment Station have been presented in five previous papers (1,[1] 2, 3, 4, 5). In those papers references were given to the work of Haigh in 1917 (6). The investigation at Annapolis included a variety of ferrous and non-ferrous metals. The experiments showed that a stress-cycle graph representing corrosion-fatigue is a curve approaching a horizontal asymptote, which may be called a "corrosion-fatigue limit." This limit may be far below the ordinary endurance limit. To avoid confusion, the term "endurance limit" was restricted to mean the fatigue limit obtained by tests in air with specimens as free as possible from stress concentration and from corrosion. The endurance limit as thus obtained presumably represents an inherent property of the metal.

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[1] Published by permission of the Secretary of the Navy.

[2] Metallurgist, U. S. Naval Engineering Experiment Station, Annapolis, Md.

The papers above mentioned gave a general survey of the behavior of metals under corrosion-fatigue. As a result of this survey the fourth paper (4) gave a tentative analysis of the corrosion-fatigue process. According to this analysis, if the initial stress is below the endurance limit but above the corrosion-fatigue limit, the corrosion-fatigue process is divided into two periods. The first period is the formation of pits. As the initial stress is below the endurance limit, such pit formation is assumed to be due to electrolytic solution pressure, not to ordinary fatigue. The electrolytic solution pressure that causes the pitting, however, is possibly not the solution pressure of stressless corrosion but an enhanced solution pressure due to the cyclic stress. Whatever may be the reason, a metal under cyclic stress behaves as if the solution pressure were increased by the cyclic stress.

As the pit progresses, both the actual stress and the effective solution pressure increase. The increase of actual stress is due to stress concentration at the bottoms of the pits. The increase of effective solution pressure is due to the increase of actual stress.

The mutual intensification of stress and solution pressure continues until the actual stress at the bottoms of the pits surpasses the endurance limit. The second period of corrosion-fatigue then begins. In this period the process is merely fatigue accelerated by the corrosion. The cracks advance at a continually accelerated rate until the specimen breaks.

PREVIOUS INVESTIGATION OF EFFECT OF CYCLIC STRESS ON CORROSION

According to this analysis of the corrosion-fatigue process, the depth and sharpness of the pits formed in the first period of corrosion-fatigue depend not only on the natural susceptibility of the alloy to corrosion pitting but also on the influence of cyclic stress on corrosion pitting. The phenomena of corrosion-fatigue seemed to indicate that cyclic stress accelerates corrosion pitting. As there was practically no quantitative information, however, about the influence of cyclic stress on corrosion, an investigation of this subject was begun at the Naval Engineering Experiment Station. Results of the investigation have been presented in three papers (7, 8, 9).

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[1] The boldface numbers in parentheses refer to the papers given in the list of references appended hereto.

These three papers discussed the effect of cyclic stress on corrosion of a number of ferrous and non-ferrous metals. The effect of corrosion under cyclic stress was estimated by observing the resultant change in the fatigue limit. The fatigue limit of the previously corroded specimen was compared with the endurance limit of the alloy. For this purpose, specimens were corroded at various cyclic stresses from zero to the endurance limit, for various times, and at various cycle frequencies. The specimens were then oiled and subjected to fatigue test at 1450 r.p.m. in air. Each experiment, therefore, consisted of two stages, a corrosion stage and a fatigue stage. The lowering of the fatigue limit due to the corrosion was used as a criterion of the depth and sharpness of the corrosion pitting.

Results of the second or fatigue-stage tests were presented in ordinary stress-cycle graphs. From the fatigue limits obtained by means of these stress-cycle graphs, several other types of graphs were derived to illustrate the effect of cyclic stress, time, and number of cycles on corrosion.

OUTLINE OF CONTINUED INVESTIGATION OF EFFECT OF CYCLIC STRESS ON CORROSION

As shown in the three papers just mentioned, damage of metal under corrosion depends on three variables (in addition to others): stress, time, and number of cycles. The object of the investigation described in the present paper was to study further the effect of these three variables on the fatigue limit. The interrelationship of four variables, therefore, is to be considered.

In investigating this subject the method of experiment was that described in the three previous papers (7, 8, 9). Each experiment consisted of two stages. In the first or corrosion stage the specimen was subjected to cyclic stress while in contact with water. In the second or fatigue stage, the corroded specimen, after being oiled, was subjected to fatigue tests at 1450 r.p.m. in air.

The term "corrosion-stress" will be used to designate the cyclic stress used in the first or corrosion stage. The term "fatigue limit" or "resultant fatigue limit" will be used to designate the fatigue limit as obtained in the second stage.

The interrelationship of corrosion stress, time, number of cycles and resultant fatigue limit, is illustrated by diagrams of various types. Three-dimensional representation of the stress-time-cycle relationship is also discussed. Parts II to V inclusive discuss the effect of stress, time and number of cycles on corrosion of carbon and ordinary alloy steels. Part VI discusses the effect of the same three variables on corrosion of duralumin. Part VII discusses the effect of these variables on corrosion of monel metal and stainless iron.

Machines and Specimens:

The rotating-cantilever machines and specimens used in most of the experiments have been described in previous papers (10, 11). The conically tapered specimen is so designed that the maximum stress is $\frac{3}{4}$ in. out from the inner fillet, and the stress varies only about 1.5 per cent over a length of 1.5 in. With this specimen, therefore, a comparatively large region may be subjected to simultaneous corrosion and cyclic stress.

TABLE I.—CHEMICAL COMPOSITION OF MATERIAL.

All values are averages of at least two determinations, usually more.

Material	Designation	Carbon, per cent	Manganese, per cent	Phosphorus, per cent	Sulfur, per cent	Silicon, per cent	Nickel, per cent	Chromium, per cent	Copper, per cent	Iron, per cent	Magnesium, per cent	Aluminum, per cent
Medium Carbon Steel	JR	0.46	0.65	0.023	0.016	0.016	....	....	....	....	....	....
Copper Steel	JK	0.27	0.47	0.010	0.029	0.01	....	....	0.82	....	....	....
Nickel Steel	IW	0.28	0.50	0.015	0.036	0.19	3.70	0.26	....	....	....	....
Chromium-Nickel Steel	BC	0.28	0.69	0.014	0.016	0.21	1.51	0.73	....	....	....	....
	AY	0.43	0.60	0.023	0.016	0.21	2.16	0.95	....	....	....	....
Stainless Iron	JB	0.11	0.41	0.012	0.026	0.11	0.43	12.71	0.06	....	....	....
Monel Metal, Cold Rolled	EP	0.16	0.95	0.015	0.017	0.05	67.51	....	29.54	1.76	....	....
Duralumin, Tempered	IN	....	0.56	....	....	0.50	....	....	3.86	0.61	0.44	94.01[a]
	JU	....	0.52	....	....	0.18	....	....	4.14	1.17	0.62	93.37[a]
	KA	....	0.63	....	....	0.21	....	....	4.26	0.67	0.63	93.60[a]

[a] By difference.

The method of alternate longitudinal and transverse polishing of specimens has been previously described (11, 12). The surface finish by this method is sufficiently smooth to permit examination of the structure at a magnification of 100. Details of machining and polishing are tabulated and kept on file at the Naval Engineering Experiment Station. This information is available for those who are interested.

TABLE II.—HEAT TREATMENTS OF MATERIAL.

Material	Designation	Temperature Heated, deg. Fahr.	Time Held, minutes	Cooled in	Temperature Reheated, deg. Fahr.	Time Held, minutes	Cooled in
Carbon Steel................	JR-W-10	1500	60	Water	1000	120	Furnace
Copper Steel................	JK-W-9	1600	60	Water	900	120	Furnace
Nickel Steel...............	IW-W-10	1450^{a}	60	Water	1000	120	Furnace
	IW-14.5	1450	60	Furnace	....	...	......
Chromium-Nickel Steel.....	BC-W-10	1550	60	Water	1000	120	Furnace
	AY-W-10	1550	60	Water	1000	120	Furnace
Stainless Iron...............	JB-W-12	1800	60	Water	1200	120	Furnace
Monel Metal, Cold Rolled....	EP-8	800	180	Furnace	....	...	......
Duralumin, Tempered......	IN			As received			
	JU			As received			
	KA			As received			

[a] Previously heated to 1675° F., held 60 minutes, cooled in air.

TABLE III.—RESULTS OF TENSION, CHARPY AND ENDURANCE TESTS: AVERAGES AND MEAN DEPARTURES FROM AVERAGES.

Unless otherwise indicated, each value is the result of two determinations, except endurance values.

Material	Condition	Designation	Tensile Strength, lb. per sq. in.	Johnson Limit, lb. per sq. in.	Proof Stress, lb. per sq. in.	Elastic Limit, lb. per sq. in.	Proportional Limit, lb. per sq. in.	Elongation in 2 in., per cent	Reduction of Area, per cent	Charpy Impact Value, ft-lb.	Endurance Limit, Rotating Cantilever lb. per sq. in.
Medium Carbon Steel	Quenched and Drawn	JR-W-10	129 300^{c} ±1 400	99 000^{c} ±2 000	100 100^{c} ±1 700	96 900^{c} ±1 900	90 600^{c} ±2 500	17.9^{c} ±0.9	54.4^{c} ±1.5	30.3 ±0.5	59 000
Copper Steel	Quenched and Drawn	JK-W-9	93 100^{d} ±1 900	71 200^{d} ±1 900	71 500^{d} ±2 000	70 900^{d} ±2 200	62 000^{d} ±4 300	25.9^{d} ±1.4	61.9^{d} ±1.6	40.0 ±0.0	49 500
Nickel Steel	Quenched and Drawn	IW-W-10	128 100^{h} ±2 800	114 000^{h} ±3 000	114 600^{h} ±3 000	110 000^{h} ±3 700	94 300^{h} ±2 600	20.6^{h} ±0.8	61.8^{h} ±0.8	29.0 ±0.0	64 000
	Annealed	IW-14.5	90 200^{g} ±1 700	60 600^{g} ±1 000	59 500^{g} ±1 600	59 400^{g} ±2 200	53 800^{g} ±6 300	29.9^{g} ±0.8	59.1^{g} ±0.9	30.9 ±0.4	49 000
Chromium-Nickel Steel	Quenched and Drawn	BC-W-10	137 700^{f} ±2 500	120 800^{f} ±3 600	122 600^{f} ±3 600	117 100^{f} ±3 200	101 900^{f} ±4 800	19.4^{f} ±0.8	53.3^{f} ±1.1	30.6 ±3.0	68 000
	Quenched and Drawn	AY-W-10	157 800^{d} ±3 100	141 400^{d} ±3 200	143 100^{d} ±3 100	136 200^{d} ±3 300	106 100^{d} ±4 200	16.8^{d} ±1.0	50.3^{d} ±1.5	16.1 ±0.6	76 000
Stainless Iron	Quenched and Drawn	JB-W-12	98 400^{d} ±1 500	68 000^{d} ±2 000	70 300^{d} ±1 500	64 200^{d} ±1 400	61 700^{d} ±1 900	24.1^{d} ±0.8	69.6^{d} ±0.2	47.6 ±0.9	50 000
Monel Metal, Cold Rolled	Annealed	EP-8	128 900^{e} ±700	94 100^{e} ±1 000	95 600^{e} ±600	88 600^{e} ±1 400	81 300^{e} ±1 700	20.4^{e} ±0.8	58.0^{e} ±0.4	53.6 ±2.6	52 000
Duralumin, Tempered	As Received	IN	58 100^{b} ±1 200	33 200^{b} ±2 200	33 700^{b} ±1 900	33 200^{b} ±2 200	21 400^{b} ±2 400	27.6^{b} ±1.4	42.1^{b} ±1.5	12.2 ±0.3	16 500
	As Received	JU	53 200^{d} ±800	25 900^{d} ±1 200	26 500^{d} ±1 100	24 400^{d} ±1 200	22 100^{d} ±1 300	24.9^{d} ±0.8	40.3 ±1.7	10.1^{a}	15 600
	As Received	KA	54 500^{i} ±1 600	26 500^{i} ±1 400	27 200^{i} ±1 300	24 800^{i} ±1 000	22 800^{i} ±1 000	23.4^{i} ±2.1	38.7^{i} ±3.3	....	......

[a] One determination.
[b] Four determinations.
[c] Six determinations.
[d] Seven determinations.
[e] Nine determinations.
[f] Eleven determinations.
[g] Nineteen determinations.
[h] Twenty-three determinations.
[i] Five determinations.

For investigation of aluminum alloys it was found desirable to use higher cycle frequency than had been used in the investigation of steel described in the most recent paper (9). For this reason a machine was developed for making rotating-cantilever tests at 10,000 r.p.m. This machine consisted of a shaft driven by belt and pulleys from a motor running at about 1800 r.p.m. Vibration of the test specimens at critical speeds caused some difficulty at first. By lightening all attachments to the end of the specimen, however, it was found possible to make tests at the desired speed using the ordinary specimen for steels and a specimen having somewhat larger diameter for duralumin. As an additional precaution to dampen vibrations, the lower end of the spring to which test loads were applied was fastened in position after the spring had been stretched by the load. The amount of extension of the spring was frequently checked by weights during the progress of long tests. Results of tests made at 10,000 r.p.m. with steel and duralumin are included in this paper.

The Corrosion Stage:

In the corrosion stage, the specimens described, while under cyclic stress, were corroded in a water stream, which was diagonally applied so as to surround with water the tapered portion of the specimen. Specimens were thus corroded under various stresses and cycle frequencies, for various times and numbers of cycles.

In these experiments two kinds of water were used, the carbonate water and the Severn River water that were used in experiments described in previous papers (2, 3, 4, 8). In Severn River water the salt content averages about one-sixth that of sea water.

Some specimens were tested after stressless corrosion by spray. The solution used in this spray was the soft water described in a previous paper (8).

Material, Chemical Composition and Heat Treatment:

Material for this investigation was obtained in the form of round rods 1 in. in diameter. The chemical composition for the alloys used is given in Table I. Details of heat treatment are given in Table II.

Tension and Impact Tests:

Results of tension and Charpy impact tests are given in Table III. In this table, "elastic limit" means the highest stress that leaves no appreciable permanent deformation after removal of the load; "proof stress" means the stress that results in a permanent deformation of 0.0001 in. per inch of length after removal of the load.

PART II.—EFFECT OF STRESS, TIME, AND NUMBER OF CYCLES ON CORROSION OF CARBON AND ORDINARY ALLOY STEELS

Extension of Field of Investigation to Include Effect of Stresses Greater Than the Corrosion-Fatigue Limit

In the first two papers dealing with the effect of stesss on corrosion (7, 8), the investigation was confined to the effect of stresses lower than the corrosion-fatigue limit. In the third paper (9), however, the field of investigation was extended to include the effect of stresses greater than the corrosion-fatigue limit. In using corrosion stresses greater than the corrosion-fatigue limit it must be kept in mind that, if corrosion be continued long enough, the first period of corrosion-fatigue will be exceeded and the second period entered. In the second period the progress of the damage is due to fatigue as well as to corrosion. To avoid this complication the corrosion stage must be stopped before, through stress concentration, the actual stress at the bottoms of the corrosion pits reaches the endurance limit of the metal. The actual stress at bottoms of pits reaches the endurance limit when the resultant fatigue limit is lowered to equality with the corrosion stress. To keep within the first period of corrosion-fatigue, therefore, the corrosion stage must be stopped before the resultant fatigue limit has been lowered below the corrosion stress.

By thus widening the field of investigation, the range of stresses included in the investigation is now several times as great as before. The use of the higher stresses has made it possible to use lower cycle frequencies. Cycle frequencies as low as 5 per hour have now been used and appreciable effects have been obtained.

Variable-Damage and Constant-Damage Diagrams

The effect of stress, time, and number of cycles in the corrosion stage is estimated by comparing the fatigue limit obtained in the second stage with the endurance limit of the metal. The term "damage" will be used to designate the amount of decrease of the resultant fatigue limit due to the corrosion. Either the damage or the resultant fatigue limit may be considered as one of the four variables under investigation.

To represent by two-dimensional diagrams the interrelationship between four variables, two of these variables (or one independent variable and the cycle frequency) must be held constant. In the Type 2 diagrams presented in three previous papers (7, 8, 9), time and number of cycles are held constant; these diagrams, therefore, represent the relationship between the two other variables, corrosion stress and resultant fatigue limit. As the resultant fatigue limit is not held constant in obtaining such diagrams, they may be called "variable damage" diagrams.

Variable-damage diagrams have one important disadvantage. The relationship between corrosion stresses, represented by such diagrams, is the relationship between initial stresses, not between actual average stresses. As the stress concentration increases during corrosion, the actual corrosion stress increases. The actual average corrosion stress, therefore, varies with the total damage. For this reason, a variable-damage diagram does not represent correctly the relationship between actual average corrosion stresses. Moreover, the form of a variable-damage diagram may be affected by varying accumulation of corrosion products in corrosion pits of varying depth.

Table IV.—Classification of Diagrams.

Kind of Diagram	Type	Letter Combination	Reference Previously Described in
Variable Damage	No. 2	$NT.SF$	7, 8, 9
	No. 5	$S\frac{N}{T}.FT$	9
Constant Total Damage	No. 6a	$F\frac{N}{T}.ST$	9
	No. 6b	$F\frac{N}{T}.SN$	9
	No. 7	$FN.ST$	9
	No. 8	$FT.SN$	9
	No. 9	$FS.NT$	9
	Three dimensional	$F.SNT$	9
Constant Net Damage	No. 10	$DS.NT$	9
	No. 11a	$D\frac{N}{T}.ST$	9
	No. 11b	$D\frac{N}{T}.SN$	9
	No. 12	$DN.ST$	..
	No. 13	$DT.SN$	..
	Three dimensional	$D.SNT$	9

In constant-damage diagrams, the resultant fatigue limit is one of the variables that is held constant. In such diagrams, therefore, the effect of the stress-concentration factor and the variable effect of the corrosion products is minimized. Actual relationship of corrosion stresses is more correctly represented by constant-damage than by variable-damage diagrams.

Classification of Diagrams

In previous papers (7, 8, 9), various types of diagrams have been presented to illustrate the relationship between the four variables under consideration. These types have been designated previously by numbers alone. It now appears desirable to designate the types by an additional, less arbitrary, method.

For use in classifying these diagrams each variable will be designated by a letter as follows:

Corrosion stress	S
Number of cycles	N
Corrosion time	T
Resultant fatigue limit	F
Net damage	D

Net damage will be discussed later. Cycle frequency, which is not an independent variable, will be designated by $\frac{N}{T}$.

Table IV gives a classification of types, also references to previous papers in which these types have been discussed. In each letter combination listed in Table IV, the letters preceding the point represent variables that are held constant in obtaining a diagram of this type. The letters following the point represent the variables whose interrelationship is illustrated by a diagram of this type.

GENERAL DESCRIPTION OF TYPE 5 DIAGRAMS ($S\,\frac{N}{T}\cdot FT$)

Type 5 diagrams for carbon and ordinary alloy steels are shown in Figs. 1, 2 and 3. Each graph represents the relationship between corrosion-time and resultant fatigue limit for a chosen stress and cycle frequency. Each series of graphs represents some one cycle frequency. Comparison of the graphs of any series, therefore, throws light on the effect of corrosion stress. The Type 5 diagram was developed for use in deriving constant-damage diagrams. Much direct information, however, may be obtained from a study of the various Type 5 series shown in Figs. 1 to 3.

Each experimental point in these figures usually represents the result obtained with a single specimen. The method of obtaining resultant fatigue limits by extrapolation of stress-cycle graphs has been illustrated and described in previous papers (7, 8). To save space, stress-cycle graphs have not been included in this paper.

All the graphs of a series have a common origin. The ordinate of this common origin represents the endurance limit of the metal (as obtained by test of uncorroded specimens). The course of each graph illustrates the decrease in resultant fatigue limit with increase in corrosion time. The uppermost graph of each series represents the damaging effect of stressless corrosion. Each other graph represents the damaging effect of corrosion under the indicated cyclic stress.

STRESSLESS CORROSION OF STEEL AS ILLUSTRATED BY TYPE 5 DIAGRAMS

As illustrated by the uppermost graph in each series, stressless corrosion of steel lowers the fatigue limit rapidly at first, but the rate of damage gradually decreases. The decrease in rate continues at least throughout the duration of the experiments, some of which lasted more than 100 days. After about 10 to 20 days the rate of damage is slow. The decrease in rate of damage with increase in corrosion time is undoubtedly due to the protective effect of accumulating corrosion products.

In Fig. 1 are illustrated the effects of stressless corrosion of carbon steel by three different media. The three methods of corrosion were: stream of carbonate water, stream of Severn River water, spray of soft water. Comparison of the three stressless corrosion graphs indicates that the stream of Severn River water and the spray of soft water are about equally damaging. The stream of carbonate water is evidently somewhat less damaging. This difference in damaging effect, however, is not prominent unless the corrosion time exceeds 10 to 20 days. As shown in a previous paper (8), with not more than 10 days' corrosion time the damaging effect of stressless corrosion is nearly the same whether soft, carbonate, or salt water be used.

EFFECT OF STRESS AS ILLUSTRATED BY TYPE 5 SERIES

The effect of stress on corrosion is well illustrated by a Type 5 series. The graphs representing various corrosion stresses and the graph representing stressless corrosion all have a common origin. As they extend to the right, however, the graphs separate. The greater the corrosion stress, the steeper is the course of the graph. Each graph representing a corrosion stress is similar in curvature to the stressless corrosion graph. As shown in a previous paper (9), however, if a Type 5 graph is extended below the point at which the ordinate is numerically equal to the corrosion stress, the curvature is reversed and the graph descends with increasing steepness to the axis of abscissas. The only graphs that can be extended so far, however, are graphs representing corrosion stresses higher than the corrosion-fatigue limit. To save space, the entire course of such graphs has not been shown in the present paper.

If the time and number of cycles are sufficiently great, very low cyclic stress is sufficient to cause appreciable lowering of the Type 5 graph below the corresponding graph representing stressless corrosion. That corrosion stress of only 5000 lb. per sq. in. has appreciable effect on the position of the Type 5 graph is illustrated in Fig. 1 by the series representing the effects of fresh and salt water at 1450 r.p.m., in Fig. 2 by the 1450 r.p.m. series, and in Fig. 3 by the lower two series. The same fact is illustrated by some of the Type 5 series in a previous paper (9). That corrosion stress of only 3000 or 4000 has appreciable effect on corrosion of carbon steel in salt water is clearly shown by a series in Fig. 1. That corrosion stress of only 3000 has appreciable effect on the corrosion of nickel steel in carbonate water is shown in the middle series of Fig. 3. The rather wide scatter of points in this series is not surprising, as the effect of unavoidable variation in corrosion conditions is large in comparison with the effect of such low corrosion stresses. The evidence seems conclusive, therefore, that cyclic stress of only 3000 lb. per sq. in. has an appreciable effect on corrosion. It seems probable that the effect of even lower corrosion stresses could be detected.

As shown by comparison of various Type 5 series obtained with the same steel, the lower the cycle frequency, the greater the corrosion stress and total time necessary to cause the Type 5 graph to descend appreciably below the graph representing stressless corrosion. The minimum corrosion stresses that cause appreciable descent of the Type 5 graph below the stressless corrosion graph are the "notching stresses" discussed in previous papers (7, 8, 9) in connection with Type 2 diagrams.

TOTAL DAMAGE AND NET DAMAGE

The total damage due to corrosion under cyclic stress is measured by the distance of the Type 5 graph (at any point) below a horizontal line through the origin of the graph. This damage, however, is not due entirely to the cyclic stress. The effect of the cyclic stress is measured by the vertical distance of the Type 5 graph (at any point) below the stressless-corrosion graph. The damage due to corrosion under cyclic stress, therefore, may be resolved into two parts. The total damage consists of the damage due to stressless corrosion plus the excess or net damage due to the cyclic stress. The ratio of net damage to total damage evidently varies with corrosion time and other corrosion conditions.

DERIVATION OF OTHER DIAGRAMS FROM TYPE 5 SERIES

Though the Type 2 graphs presented in previous papers (7, 8, 9) were not obtained from Type 5 graphs, they could have been so obtained. The intersections of the Type 5 graphs of any series with a chosen vertical line could be used to obtain Type 2 graphs. The intersections of the Type 5 graphs of one or more series with a chosen horizontal line may be used to obtain constant-damage graphs. Constant-damage graphs are discussed in Part III.

PART III.—CONSTANT DAMAGE DIAGRAMS FOR CARBON STEELS, CARTESIAN COORDINATES

DERIVATION OF CONSTANT-DAMAGE DIAGRAMS

A constant-damage graph is so drawn that the coordinates of any point in the graph represent the stress, time and number of cycles necessary to reduce the fatigue limit to a chosen constant value. From Fig. 1, for example, it is possible to derive constant-damage graphs representing corrosion conditions necessary to reduce the fatigue limit to 35,000 lb. per sq. in. To do this the intersections of the Type 5 graphs with a horizontal line representing a resultant fatigue limit of 35,000 lb. per sq. in. would be used.

Constant-damage diagrams of various types are discussed in Parts III and IV. In obtaining these diagrams the Type 5 diagrams were plotted on scales that were most suitable for the purpose. The diagrams in Figs. 1 to 3 are merely illustrative and are not necessarily on the same scales that were used in obtaining the constant-damage diagrams discussed in Parts III and IV.

TYPE 9 DIAGRAM (*FS.NT*)

To obtain a constant-damage diagram, the resultant fatigue limit is held constant. By holding one other factor or the ratio between two other factors constant, several types of constant-damage diagrams may be obtained. By holding the corrosion-stress constant, Type 9 diagrams are obtained.

Figure 8 shows a series of Type 9 graphs obtained with a heat-treated carbon steel. Each graph represents the times and numbers of cycles (with the indicated constant corrosion stress) necessary to reduce the fatigue limit to the indicated constant value. Not only is each graph a constant-damage graph, but the entire series is a constant-damage series. Each graph represents the conditions necessary to reduce the fatigue limit to 35,000 lb. per sq. in., about 60 per cent of the endurance limit.

In a Type 9 diagram, lines radiating from the origin of coordinates are constant-frequency lines. The range of frequencies represented in Fig. 8 is from 10,000 to 1.5 cycles per minute.

As shown in the figure the Type 9 graph at each end is apparently approaching the coordinate axis, or a line parallel to the coordinate axis, as asymptote. If extended far enough to the right, however, the graph reverses its curvature as it approaches the "time boundary."

The line marked "time boundary" in Fig. 8 represents, by its constant abscissa, the time necessary to reduce the fatigue limit to 35,000 lb. per sq. in. by stressless corrosion. This time has been obtained by extrapolating the Type 5 stressless corrosion graph. Obviously the Type 9 graphs cannot extend to the right of the time boundary. These graphs are represented in Fig. 8 as reversing their curvature and converging to the axis of abscissas at its junction with the time boundary. That this is a qualitatively correct representation will be made clear by study of Type 9 graphs on a logarithmic scale to be presented in Part IV.

The lower the corrosion stress, the more distant is the Type 9 graph from the origin of coordinates. With decrease in corrosion stress, the effect of this decrease on the position of the Type 9 graph becomes greater. The distance between the 8000 and 7000-lb.-per-sq.-in. graphs, for example, is much greater than the distance between the 20,000 and 15,000-lb.-per-sq.-in. graphs.

THREE-DIMENSIONAL CONSTANT-DAMAGE DIAGRAM

The Type 9 series shown in Fig. 8 may best be studied by imagining it to be the top view of a three-dimensional diagram representing the effect of stress, time and number of cycles on corrosion. If an axis of corrosion stresses be imagined perpendicular to the plane of Fig. 8 at the origin of coordinates, the Type 9 graphs in this figure would be viewed as contour lines on a constant-damage surface. This constant-damage surface evidently slopes downward as it recedes from the vertical axis of corrosion-stresses. With decrease in corrosion stress, as indicated by the increasing distance between contour lines, the slope decreases. With decrease in the corrosion stress below about 8000 lb. per sq. in. the constant-damage surface, on the scale represented in Fig. 8, evidently becomes nearly horizontal.

Figure 9 is a photograph of a three-dimensional model illustrating constant-damage relationship for a nickel steel, IW-W-10. The direction of view shown in Fig. 9 is toward the vertical axis of corrosion stresses. The horizontal axes on the right and left respectively in the figure are the axis of cycles and the time axis. On the right and left respectively are the stress-cycle plane of reference and the stress-time plane of reference. The heavy contour lines in the figure correspond to the Type 9 graphs shown in Fig. 8. The radiating constant-frequency lines in Fig. 8 correspond to the radiating sloping curves in Fig. 9. These sloping curves when projected on either of the vertical planes of reference become two-dimensional diagrams which have been designated Type 6.

TYPE 6 (*a*) SERIES $\left(F\frac{N}{T}\cdot ST\right)$

If the sloping constant-frequency lines in Fig. 9 be viewed in a direction parallel to the axis of cycles, they would be projected on the stress-time plane of reference. A two-dimensional diagram representing this point of view has been called a Type 6 (*a*) series. Such a series, obtained with the same carbon steel that was used in obtaining Fig. 8, is shown in Fig. 10. As shown in this figure, all the graphs are on the same time scale, but each graph has its own cycle scale. In the Type 6 (*a*) series the low-frequency graphs are uppermost.

The Type 6 (*a*) graph is similar in form to the Type 9 graph. If extended far enough to approach the time boundary, the Type 6 (*a*) graph reverses its curvature and descends abruptly to the axis of abscissas. This abrupt reversal of curvature is another aspect of the "notching stress" that has been discussed in three previous papers (7, 8, 9) in connection with Type 2 graphs.

TYPE 6 (*b*) SERIES $\left(F\frac{N}{T}\cdot SN\right)$

If the sloping constant-frequency lines in Fig. 9 be viewed in a direction parallel to the time axis, they would be projected on the stress-cycle plane of reference. A two-dimensional diagram representing this point of view has been called a Type 6 (*b*) series. Figure 11 represents a Type 6 (*b*) series obtained with the same carbon steel that is represented in Figs. 8 and 10. As shown in Fig. 11, all the graphs are on the same cycle scale, but each graph has its own time scale. In the Type 6 (*b*) series the high-frequency graphs are uppermost.

TYPE 7 DIAGRAM (*FN.ST*) AND THE TYPE 8 DIAGRAM (*FT.SN*)

On the constant-damage surface in Fig. 9 are coordinate lines parallel either to the stress-time or stress-cycle plane of reference. The former will be called stress-time lines, the latter will be called stress-cycle lines. If the stress-time lines be projected on the stress-time plane of reference they would form a two-dimensional series which has been called Type 7. If the stress-cycle lines be projected on the stress-cycle plane of reference, they would form a two-dimensional series, which has been called Type 8. Type 7 and Type 8 series are shown in a previous paper (9). The Type 7 and Type 8 graphs are similar in form to the Type 6 and Type 9 graphs. Type 7 graphs, if extended so far that they approach the time boundary, reverse their curvature as do the Type 6 and Type 9 graphs. Type 8 graphs, however, are parallel to the time boundary and hence have no reversal of curvature.

The characteristics of the above described constant-damage graphs and surfaces will be considered more in detail in Part IV.

PART IV.—CONSTANT-DAMAGE DIAGRAMS, LOGARITHMIC COORDINATES—CARBON AND ORDINARY ALLOY STEELS

Type 9 Diagrams (*FS.NT*), Logarithmic Coordinates

On a Cartesian scale, the curvature of the Type 9 graph is so great that interpolation and extrapolation are difficult. Moreover the range of coordinates and range of cycle frequencies that can be represented in one series are much less than is desirable. On a logarithmic scale, however, the curvature of the Type 9 graph is greatly reduced. The lines of constant frequency, which on a Cartesian scale are radiating lines, on a logarithmic scale become parallel diagonal lines. It is thus possible to include in one constant-damage series a wide range of corrosion stresses and cycle frequencies.

In Figs. 12 and 13 are three Type 9 series obtained with the same carbon steel that is represented in Figs. 8, 10 and 11. In these figures the range of actual cycle frequencies represented is from 10,000 cycles per minute to 5 cycles per hour. Additional constant-frequency lines, however, have been added wherever possible to represent 1 cycle per hour and 1 cycle per day. These lines are designated 1 revolution per hour and 1 revolution per day, respectively.

Type 9 Constant-Damage Series, Carbon Steel JR-W-10

The series of graphs in Fig. 12 represents corrosion conditions necessary to reduce the fatigue limit to 35,000 lb. per sq. in. in fresh water. The two series in Fig. 13 represent conditions necessary to reduce the fatigue limit to 30,000 lb. per sq. in. in fresh and salt water, respectively. The two series in Fig. 13, therefore, represent greater damage than the series in Fig. 12.

In drawing the graphs of these three series, and other series to be described, consideration has been given to each series as a whole. Each graph has been based not merely on its own experimental points but on the most probable relationship to neighboring graphs.

The "boundary graph" of each series is a graph obtained with corrosion stress equal to the resultant fatigue limit. If higher corrosion stresses were used, the actual stress at the bottoms of corrosion pits would exceed the endurance limit before the resultant fatigue limit would be reduced to the desired value. The other boundary of each series is the time boundary. This line has the same significance as in Figs. 8 and 10. It represents the time necessary to reduce the fatigue limit to the chosen value by stressless corrosion. The positions of the time boundary in Figs. 12 and 13 have been determined only approximately by long extrapolation of the Type 5 graphs shown in Fig. 1.

The Type 5 graphs were extrapolated on the assumption that throughout the long time represented by the time boundaries in Figs. 12 and 13, the rate of damage continually decreases. As shown in Figs. 1 to 3, the decrease in rate of damage continues for more than 100 days. It is possible, however, that after a longer corrosion time the rate of damage may no longer decrease and may even begin to increase.

The curvature of the Type 9 logarithmic graphs is qualitatively similar to the curvature of these graphs on a Cartesian scale. The curvature is slight, however, unless the graphs are extended so far to the right that they approach the time boundary of the series. All the graphs of a series as they approach the time boundary, reverse their curvature, converge toward the time boundary, and approach the axis of abscissas. This converging approach has not been exactly determined, but the representation in Figs. 12 and 13 is qualitatively correct.

The graphs of a series are usually not far from parallel unless they are extended so far that they approach the time boundary. The distance between adjacent graphs, for equal differences in corrosion stress, gradually increases as the corrosion stress is lowered. Change in position of the Type 9 graph with change in corrosion stress is proportional to percentage change, rather than in proportion to actual change of corrosion stress.

It is advantageous to consider these Type 9 graphs as originating in the time boundary. The course of each graph, as it turns to the left from the time boundary, represents the effect of increasing number of cycles (or the effect of increasing cycle frequency) in accelerating the damage due to corrosion. As illustrated by an entire series of graphs, the lower the corrosion stress, the higher the cycle frequency necessary to accelerate appreciably the damage of metal under corrosion. High corrosion-stress has appreciable effect even if the cycle frequency be very low. The abruptness with which the Type 9 graph turns to the left varies with the corrosion stress. Graphs representing high corrosion-stresses curve to the left gradually; graphs representing lower corrosion stresses leave the time boundary more abruptly.

Form and Position of the Type 9 Series as Affected by the Amount of Damage Represented:

The series represented in Fig. 12 and the series on the left of Fig. 13 are each fresh water series obtained with the same steel. The damage represented by the latter series is greater than the damage represented by the former series, as the resultant fatigue limit for the latter series is lower than for the former. It will be observed that the graphs in the two series are similar in form and in direction. As would be expected, however, the graphs in the series representing greater damage are further from the origin of coordinates than are the corresponding graphs of the other series.

Comparison of Fresh and Salt-Water Series:

Comparison of the two series in Fig. 13 shows that the form and direction of the fresh-water and salt-water graphs are not very different. As would be expected, the fresh-water graphs are further from the origin of abscissas than are the corresponding salt-water graphs. The difference in position between corresponding graphs, however, is not appreciable except in the parts of the graphs that represent short corrosion time. This subject needs further investigation.

TYPE 9 CONSTANT-DAMAGE SERIES, COPPER STEEL JK-W-9

The graphs shown in Fig. 14 were obtained with a copper steel that had been quenched and tempered. The resultant fatigue limits of the upper and lower series are 30,000 and 35,000 lb. per sq. in., respectively. The upper series, therefore, represents greater damage than the lower series. The time boundaries for both series have been accurately determined.

The graphs in Fig. 14 are similar in form and direction to the graphs in Figs. 12 and 13. As would be expected, the graphs in the upper series of Fig. 14 are further from the origin of coordinates than the graphs in the lower series.

The positions of the series in Fig. 14 should be compared with the fresh-water series in Figs. 12 and 13 on the basis of equality in stress-concentration ratio rather than on the basis of equality of resultant fatigue limits. The series in Fig. 12 and the upper series in Fig. 14 represent unequal resultant fatigue limits, but nearly equal stress-concentration ratios. It will be observed that these two series differ only slightly in position. This confirms the conclusion expressed in a previous paper (9) that the position of the Type 9 series depends chiefly on the stress-concentration ratio rather than on differences in chemical composition that are not accompanied by appreciable differences in corrosion resistance. It was also shown in the previous paper that the position of the Type 9 series is little affected by changes in physical properties due to heat-treatment.

TYPE 6 (*a*) DIAGRAMS $\left(F\frac{N}{T}.ST\right)$, LOGARITHMIC COORDINATES

In Figs. 17 and 18 are six series of Type 6 (*a*) graphs representing the same materials that are represented in Figs. 12 to 14 by Type 9 graphs. Abscissas in Figs. 17 and 18 are measured from right to left. This arrangement is made so that each series together with the corresponding Type 9 series may represent two views of lines on a constant-damage surface (constant-damage surfaces will be discussed later).

In drawing the graphs in Figs. 17 and 18, each series has been considered as a whole, and the graphs have been drawn to represent the most probable interrelationship.

The series in Figs. 17 and 18 should be compared with the series of the same type drawn on an ordinary coordinate scale, as shown in Fig. 10. The curvature of the graphs on a logarithmic scale is greatly reduced. A few of the high-frequency graphs in Figs. 17 and 18 show slight curvature, but most of the graphs in these figures are nearly straight lines. The general impression obtained from a study of the six series is that the representative Type 6 graph on a logarithmic scale is a sloping nearly straight line, except as it approaches the time boundary.

It is advantageous to view each series as originating in the time boundary. The course of each graph as it extends to the right from this boundary represents the effect of increasing corrosion stress in decreasing the corrosion time necessary to cause constant damage.

THREE-DIMENSIONAL CONSTANT-DAMAGE DIAGRAMS, LOGARITHMIC COORDINATES

Each Type 6 series in Figs. 17 and 18 may best be studied in connection with the corresponding Type 9 series in Figs. 12 to 14. Each Type 9 series may be imagined as the top view of a constant-damage surface, in which the Type 9 graphs are contour lines. This surface slopes diagonally downward as it recedes from the imagined vertical axis of corrosion stresses. The corresponding Type 6 (*a*) series represents this sloping surface as viewed in a direction parallel to the axis of cycles. The Type 6 (*a*) graphs are the same constant-frequency lines that appear in the corresponding Type 9 series as parallel diagonal lines.

The constant-damage surface on a logarithmic scale (except as it approaches the time boundary) is evidently a shallow trough or nearly plane surface sloping diagonally downward as it recedes from the axis of corrosion stresses. As the constant-damage surface approaches the time boundary, it curves abruptly downward and approaches the vertical time-boundary plane as an asymptote.

That the trough is shallow will be evident when it is considered that the Type 9 graphs, which are only slightly curved, are diagonally-cut sections of this trough. The curvature of the Type 9 graphs in most of the series shown in a previous paper (9), moreover, is somewhat less than in the series shown in Figs. 12 and 13. The stress-time-cycle relationship for constant damage, therefore, is an exponential relationship, except when the corrosion time is so great that the net damage is only a small part of the total damage.

PART V.—CONSTANT NET-DAMAGE DIAGRAMS, CARBON AND ORDINARY ALLOY STEELS

Total Damage and Net Damage.—The relation between total damage and net damage, as illustrated by Type 5 diagrams, was discussed in Part I. It was there shown that the total damage due to corrosion under cyclic stress may be resolved into two parts. The first part is the damage that would be caused in the same time by stressless corrosion, the second part is the excess or net damage due to the cyclic stress. In corrosion of ordinary steels the first or stressless-corrosion part is relatively large. Under some conditions of stress, time and number of cycles, the net damage may be only a small part of the total damage.

Constant Net-Damage Diagrams

The effect of cyclic stress on corrosion is measured by net damage rather than by total damage. The relation of net damage to corrosion stress, time and number of cycles, therefore, is of theoretical and practical importance. This relationship is best illustrated by constant net-damage diagrams.

By "constant net-damage" is meant a constant percentage lowering of the fatigue limit below the fatigue limit that would result in the same time from stressless corrosion. To represent constant net-damage, therefore, a line may be drawn in a Type 5 diagram so that its ordinates will be a constant percentage of the corresponding ordinates of the stressless-corrosion graph. The broken line in the middle series of Fig. 3 has been so drawn. At any point in this line, the ordinate is 85 per cent of the corresponding ordinate of the stressless-corrosion graph. The coordinates of the intersections of this line (and other similar lines) with the Type 5 graphs may be used in plotting graphs to represent the stress-time-cycle relationship for constant net damage.

TYPE 10 CONSTANT NET-DAMAGE DIAGRAMS ($DS.NT$)

The Type 10 diagram is similar in arrangement of coordinates to the Type 9 diagram. In Figs. 19 and 20 are shown four series of Type 10 graphs. The two series in Fig. 19 represent the same carbon steel that is represented by Figs. 8, 10, 11, 12 and 13. The series on the left and right respectively illustrate the effect of stress, time and number of cycles on corrosion of this steel in fresh and salt water. The upper series in Fig. 20 represents the copper steel that is represented in Fig. 14. The lower series represents a heat-treated nickel steel. A Type 10 series for this steel was shown in a previous paper (9). The series in the present paper includes results of additional experiments.

On account of the wide scatter of experimental points in Figs. 19 and 20 no attempt has been made to draw interpretative graphs. The experimental points have been connected by straight lines. A study of the four series, however, gives clear indication that, in spite of the apparent irregularity of form of the individual graphs, the ideal Type 10 graphs are nearly parallel straight lines. The angle between these ideal graphs and the axis of abscissas is evidently less than the corresponding angle between the Type 9 graphs and the same axis. In most of the Type 10 series for ordinary steels, shown in this paper and a preceding paper (9), the graphs representing low corrosion stresses are nearly parallel to the axis of abscissas. For ordinary steels, therefore, time is a less influential factor in net damage than in total damage. Net damage for these steels depends chiefly on stress and number of cycles. With low corrosion stresses, net damage appears to be almost, if not quite, independent of time.

TYPE 11 (*a*) CONSTANT NET-DAMAGE DIAGRAMS $\left(D\frac{N}{T}.ST\right)$

The Type 11 (*a*) diagram is similar in arrangement of coordinates to the Type 6 (*a*) diagram. In Fig. 22 the upper three series represent the same steels that are represented by Type 10 graphs in Figs. 19 and 20.

The scatter of experimental points is much less for the Type 11 (*a*) than for the Type 10 graphs. (The reason for this will be understood if the Type 10 and Type 11 (*a*) series are studied as two views of a three-dimensional diagram.) In drawing the graphs of these three series, consideration has been given to each series as a whole. Each series is best represented by straight nearly parallel lines as shown in Fig. 22. Not only are the lines throughout each series nearly parallel, but the lines of all three series are not far from parallel although one is a salt-water series and the other two are fresh-water series. The angle between the Type 11 (*a*) graphs and the axis of abscissas is less than the angle between the Type 6 (*a*) graphs and the axis of abscissas.

THREE-DIMENSIONAL CONSTANT NET-DAMAGE DIAGRAMS

Each Type 10 series in Figs. 19 and 20 may be imagined as the top view of a constant net-damage surface, in which the Type 10 graphs are contour lines. Each corresponding Type 11 (*a*) series is a front view of constant-frequency lines in the same surface.

As represented by these two views, the constant net-damage surface on a logarithmic scale is a nearly plane surface, sloping downward as it recedes from the vertical axis of corrosion stresses. The slope of this surface is considerably less than the slope of the nearly plane part of the constant total-damage surface. This explains why the Type 10 graphs are more irregular than the Type 9 graphs, and much more irregular than the Type 11 graphs. As indicated by the direction of the Type 10 graphs, the direction of greatest slope of the three-dimensional diagram makes a very small angle with a vertical plane through the axis of cycles, and at lower stresses may be almost parallel to this vertical plane.

For carbon and ordinary alloy steels, therefore, the stress-time-cycle relationship for constant net-damage is an exponential relationship, which extends over a wide range of stress, time, and number of cycles. When extended to stresses as low as 3000 lb. per sq. in. the three-dimensional diagram shows little if any tendency to curve outward toward a horizontal asymptote. There is no evidence of a stress limit below which the net effect of cyclic stress abruptly diminishes. The evidence seems to indicate that any stress cycle, however small the stress range, has an effect in increasing the damage of steel under corrosion.

Effect of Chemical Composition, Physical Properties, and Corrosion Conditions on the Position and Slope of the Three-Dimensional Diagram:

Composite Type 11 (*a*) graphs for all the ordinary steels that have been discussed in this paper and a preceding paper (9) are shown in

Fig. 23. Each diagram in this figure is a composite diagram representing a single cycle frequency. Results obtained with salt water are included with results obtained with fresh water.

These composite graphs show surprisingly little scatter of experimental points, although each graph represents results of experiments with carbon and ordinary alloy steels having a wide range of composition and physical properties, and some of the graphs represent results obtained with two kinds of water. These results seem to indicate that the net damage due to stress, time and number of cycles is practically unaffected by ordinary variations in chemical composition or physical properties, and is practically the same whether the corrosion is in fresh or salt water. The results are in accord with the fact that the notching stress for ordinary steels, as illustrated by Type 2 graphs in two previous papers (8, 9) is practically unaffected by variations in chemical composition and physical properties. The results are also in accord with the fact that the position of the Type 9 graph, as illustrated in Part IV, depends chiefly on stress-concentration ratio and not on chemical composition and physical properties.

For all ordinary steels, therefore, the net effect of stress, time, and number of cycles can be represented by practically the same three-dimensional diagram. This diagram on a logarithmic scale is a sloping nearly plane surface.

PART VI.—EFFECT OF STRESS, TIME AND NUMBER OF CYCLES ON CORROSION OF DURALUMIN

Difficulty in Investigation of Effect of Cyclic Stress on Corrosion of Duralumin

Quantitative investigation of the effect of cyclic stress on corrosion is more difficult for duralumin than for ordinary steels. In previous papers (4, 7, 8) it was pointed out that stressless corrosion apparently causes nearly as much damage to duralumin as does corrosion under cyclic stress below the corrosion-fatigue limit. This apparent lack of effect of cyclic stress was attributed to the fact that for duralumin the effect of stressless corrosion is relatively large, and thus tends to obscure the effect of cyclic stress. The low ratio of net damage to total damage for duralumin, therefore, makes it difficult to investigate quantitatively the effect of cyclic stress on corrosion.

The difficulty is increased by the fact that the scatter in a stress-cycle fatigue graph for duralumin, whether or not there has been previous corrosion, is relatively great. For ordinary fatigue, as shown by the author in several papers (3, 4, 14), the stress-cycle graph is best represented by a relatively broad band rather than by a single curved line. For previously-corroded specimens also the stress-cycle graph is best represented by a relatively broad band.

The difficulty of investigation of duralumin is further increased by the fact that, on account of the relatively low fatigue limit, the range of stresses that can be investigated is relatively small.

The ratio of net damage to total damage increases with increase in cycle frequency. For this reason, it appeared desirable in the investigation of duralumin to use the highest possible cycle frequency. A rotating-cantilever machine was, therefore, developed for experiments at 10,000 r.p.m.

EFFECT OF STRESS, TIME AND NUMBER OF CYCLES AS ILLUSTRATED BY TYPE 5 DIAGRAMS $\left(S\frac{N}{T}.FT\right)$

Type 5 diagrams for duralumin are shown in Fig. 4. The upper series of this figure shows results obtained by stressless corrosion in comparison with results obtained at various corrosion stresses and cycle frequencies. The stressless-corrosion results here shown represent three lots of duralumin differing very little in physical properties. With the exception of the individual results indicated in the legend, all the results were obtained with duralumin JU. The solid circles, triangles and squares represent results obtained by stressless corrosion. The other experimental points represent results obtained at indicated corrosion stresses.

The stressless-corrosion graph is represented by a band with broken lines as upper and lower limits. At the origin, the width of this band represents the range of endurance limits as obtained by ordinary fatigue tests in air. As the band extends to the right it probably becomes wider. As here drawn, however, the band is possibly somewhat broader than if it represented only a single lot of material. The solid line, representing average stress-cycle relationship, has been drawn to conform not only to the stressless-corrosion points in Fig. 4, but also to conform to the positions of the time boundary in the four series of Type 9 graphs in Fig. 15.

The position of the stressless-corrosion band, and the average line, in Fig. 4 should be compared with the positions of the points representing results obtained at various stresses and cycle frequencies. It will be observed that most of the small circles, representing results obtained at $1\frac{1}{2}$ r.p.m., are within the stressless-corrosion band, and one is above the average line. The only points that fall below the stressless-corrosion band are two points representing results of corrosion for 23 and 30 days.

At 1½ r.p.m., therefore, it requires more than 20 days' corrosion time to depress the resultant points below the stressless-corrosion band. At 50 r.p.m. it requires lower corrosion stress and shorter corrosion time to cause similar depression of the resultant points below the stressless-corrosion band. Further increase in frequency to 1450 and 10,000 r.p.m. causes further decrease in the minimum corrosion stress and corrosion time necessary to show unmistakably the net damage due to cyclic stress. The upper series in Fig. 4, therefore, illustrates the advantage of using as high cycle frequency as possible.

The average line, shown in the upper series in Fig. 4, has been repeated in the other series of this figure. Each of these series represents a single cycle frequency. A comparison of the graphs of a series shows the effect of corrosion stress on the position of the Type 5 graph.

The Type 5 graph for duralumin is similar in form to the Type 5 graph for steels. The form of the graph indicates that, with increasing corrosion time, the rate of damage continually decreases. The Type 5 graph for duralumin, therefore, is a "retarded-damage" graph.

TYPE 9 CONSTANT-DAMAGE SERIES, DURALUMIN

Type 9 constant-damage graphs, obtained from the Type 5 series just described, are shown in Fig. 15. The four series shown in this figure differ in resultant fatigue limit and hence in position of the time boundary.

The Type 9 series for duralumin is evidently similar in form to the Type 9 series for steels. Divergence of graphs from the time boundary, however, is at higher frequency for duralumin than for steels. At cycle frequency of only 1½ r.p.m. and even at 50 r.p.m. the breadth of the duralumin series is small. At cycle frequency of 10,000 r.p.m. the divergence of the duralumin series is no greater than the divergence of a steel series at cycle frequency of about 1 to 10 r.p.m. This illustrates the advantage of using high cycle frequency in investigation of duralumin.

As indicated by these diagrams, cyclic stress of only 4000 lb. per sq. in. has appreciable effect in accelerating damage of duralumin due to corrosion. The effect of even lower stress could probably be detected if the corrosion time were considerably increased. On account of the small ratio of net damage to total damage, however, a large number of such experiments would be necessary.

TYPE 6 (*a*) SERIES, DURALUMIN

The four series on the right in Fig. 18 are Type 6 (*a*) series corresponding to the four series that have just been described. Each

Type 9 series with its corresponding Type 6 (*a*) series should be studied as top and front view respectively of a constant-damage surface.

If higher corrosion stresses could be used so as to extend upward the two low frequency graphs of each Type 6 (*a*) series, these graphs would probably become nearly parallel to the two high-frequency graphs. Only the two graphs of highest frequency, in each series, diverge far enough from the time boundary to give clear indication of the form of the constant-damage surface. As represented by these two graphs, the constant-damage surface, for duralumin as for steels, is a diagonally sloping nearly plane surface.

TYPE 11 (*a*) CONSTANT NET-DAMAGE SERIES, DURALUMIN

The Type 11 (*a*) series for duralumin is shown in Fig. 22. The graph representing 10,000 r.p.m. shows slight scatter. The graph representing 1450 r.p.m. shows somewhat more scatter. For graphs representing lower cycle frequencies the scatter is so great that many more experiments would be necessary to determine the position. Quantitative conclusions, therefore, must be based almost entirely on the graphs representing 10,000 and 1450 r.p.m.

For duralumin as for steels, the Type 11 (*a*) series is best represented by nearly parallel straight lines. The slope of these lines is less than the slope of the Type 11 (*a*) graphs for steels (as illustrated by the upper three series in Fig. 22).

TYPE 10 CONSTANT NET-DAMAGE SERIES, DURALUMIN

In Fig. 21 the series on the left is a constant net-damage series (*DS.NT*) for duralumin. The scatter of a Type 10 graph is naturally greater than that of a Type 11 (*a*) graph. This has been discussed in Part V. The scatter of the graphs in Fig. 21 representing cycle frequencies 50 r.p.m. and 1½ r.p.m. is so great that conclusions about the form and direction of the ideal Type 10 graph must be based almost entirely on results obtained at 10,000 and 1450 r.p.m.

The evidence seems to indicate, however, that the ideal Type 10 graph is a straight line. The angle between this line and the axis of abscissas is evidently greater for duralumin than for steel. Time, therefore, is an important factor in net damage of duralumin under cyclic stress. This conclusion would also be reached from a study of the Type 5 series in Fig. 4.

THE THREE DIMENSIONAL CONSTANT NET-DAMAGE DIAGRAM, DURALUMIN

The Type 10 and Type 11 (*a*) series just described may be imagined as top and front views respectively of a constant net-damage surface. This surface, for duralumin as for steels, is probably a nearly plane surface, sloping diagonally downward as it recedes from the vertical axis of corrosion stresses. The slope of the surface for duralumin is evidently less than for steels.

For duralumin, as for steels, therefore, the stress-time-cycle relationship for constant net-damage is an exponential relationship. The diagrams here presented seem to indicate also that for duralumin, as for ordinary steels, cyclic stress, however small the range, accelerates corrosion.

PART VII.—EFFECT OF STRESS, TIME AND NUMBER OF CYCLES ON CORROSION OF MONEL METAL AND STAINLESS IRON

Range of Stresses Available:

In investigating the effect of cyclic stress on corrosion of monel metal and stainless iron, the stresses used must ordinarily be above the corrosion-fatigue limit. As shown in previous papers (7, 8) the corrosion-fatigue limit corresponds to a fairly definite pitting limit, below which the effect of corrosion stress is inappreciable, at least for the duration of corrosion time reported in those papers (7, 8).

Monel Metal

Type 5 Diagrams $\left(S\frac{N}{T}.FT\right)$:

In the investigation of monel metal it was soon noticed that specimens tested at high frequency tended either to break in the first stage of a two-stage experiment, or to show practicly no reduction of fatigue limit in the second stage. The reason for this erratic behavior became evident when the nature of the curvature of the Type 5 graph was discovered. The Type 5 graphs for monel metal, as shown in Figs. 5 and 6, are very different in form from the Type 5 graphs for ordinary steels and for duralumin. Type 5 graphs for ordinary steels and for duralumin as they extend to the right have continually decreasing slope. Type 5 graphs for monel metal, however, have continually increasing slope. This is clearly indicated by the evidence presented in Figs. 5 and 6, in spite of the rather wide scatter of individual points. The Type 5 graphs for ordinary steels and duralumin, therefore, represent continually retarded damage. The Type 5 graphs for monel metal represent continually accelerated damage.

The stressless-corrosion graph for monel metal, like the graphs representing effects of cyclic stresses, is an accelerated-damage graph. This is established by results of stressless-corrosion experiments. Moreover, if the stressless-corrosion graph were not drawn with continually increasing slope, part of the graph would be below a number of experimental points representing results of corrosion under various cyclic stresses.

That the stressless-corrosion graph is an accelerated-damage graph cannot be attributed to the effect of increasing stress concentration. Accelerated damage, however, might be caused by concentration cells due to diminished oxygen content in corrosion pits. That regions of diminished oxygen content are often regions of accelerated corrosion pitting has been shown by U. R. Evans (13). Whether concentration cells are the cause of the continually accelerated damage represented by the Type 5 graphs for monel metal cannot yet be decided. For ordinary steels and duralumin corroded in a water stream, however, the effect of concentration cells is small in comparison with the protective effect of accumulating corrosion products.

Derivation of Constant-Damage Graphs from Type 5 Series:

By selecting a resultant fatigue limit that is high enough to avoid the steepest portion of the Type 5 graphs and yet far enough below the endurance limit to represent important damage, it has been possible to obtain constant-damage graphs for monel metal. The resultant fatigue limit selected was 45,000 lb. per sq. in. As the endurance limit of the metal is about 52,000 lb. per sq. in. the stress-concentration ratio represented by the constant-damage graphs for monel metal is much smaller than the ratio represented by the graphs for steels and duralumin shown in Figs. 13 to 15.

Type 9 Constant-Damage Series (FS.NT):

In Fig. 16 is shown a constant-damage series obtained with monel metal. The reversal of curvature, so prominent in the Type 9 graphs for ordinary steels, is not found in the Type 9 graphs for monel metal. Each graph for monel metal, throughout its entire length, curves gradually toward the axis of abscissas. This difference between the Type 9 graphs for steels and for monel metal is evidently due to the above-described difference in form between the Type 5 graphs.

Type 9 graphs for monel metal are further from the origin of coordinates than are the corresponding graphs for steels, in spite of the fact that the graphs for monel metal represent much lower stress-concentration ratio than do the graphs for steels.

Type 6 (a) Constant-Damage Series $\left(F\frac{N}{T}.ST\right)$:

In Fig. 17 is shown a Type 6 (*a*) series for monel metal. This series resembles somewhat the Type 6 (*a*) series for steels as shown in

Figs. 17 and 18, although the Type 6 (*a*) graphs for monel metal diverge as they extend to the right. The slope of the nearly straight portion of the graph is somewhat less for monel metal than for steels.

Three-Dimensional Constant-Damage Diagram:

It is advantageous to consider the Type 9 and Type 6 (*a*) series for monel metal as the top and front views respectively of a constant-damage surface. This surface somewhat resembles the constant-damage surface for ordinary steels, differing chiefly in slope and in extent of the nearly straight portion.

Type 10 Constant Net-Damage Diagram (DS.NT):

For monel metal, because its Type 5 graphs are accelerated-damage graphs, it has not been found possible to derive constant net-damage graphs that are entirely free from the variable influence of stressless corrosion. For such metals, the effect of stressless corrosion increases at an increasing rate with increase in corrosion time, and finally obscures the effect of cyclic stress. In Figs. 21 and 22, therefore, the diagrams, though called "constant net-damage" diagrams, are not entirely free from the influence of stressless corrosion.

For the Type 10 series in Fig. 21 interpretative graphs have not been drawn. The evidence seems to indicate, however, that the representative Type 10 graph is a nearly straight line. The convergence of the graphs of this series as they extend to the right is probably due to the fact that the variable influence of stressless corrosion cannot be entirely eliminated when the Type 5 graph is an accelerated damage graph.

Type 11 (a) Constant Net-Damage Series $\left(D\frac{N}{T}.\,ST\right)$:

In Fig. 22 is shown a Type 11 (*a*) series for monel metal in comparison with Type 11 (*a*) series for ordinary steels and for duralumin. The Type 11 (*a*) graph for monel metal, like the Type 11 (*a*) graph for steels and for duralumin, is evidently a nearly straight line. The convergence of the monel metal graphs as they extend to the left is due, in part at least, to the fact that the variable influence of stressless corrosion cannot be entirely eliminated, when the Type 5 graph is an accelerated-damage graph.

Constant Net-Damage Surface:

The Type 10 series may be imagined as the top view of a constant net-damage surface. The Type 11 (*a*) series would then represent the front view of the same surface. The three-dimensional diagram for monel metal is less nearly plane than the three-dimensional diagram for steels. Part of this departure from planeness is probably due to the fact that the influence of stressless corrosion on the monel metal diagram has not been entirely eliminated. In view of the straightness of the Type 10 and Type 11 (*a*) graphs, however, it appears evident that the stress-time-cycle relationship for constant net-damage is an exponential relationship.

STAINLESS IRON

Type 5 Diagrams $\left(S\frac{N}{T}.\,FT\right)$:

In Fig. 7 are shown Type 5 diagrams for stainless iron. These graphs evidently resemble the graphs for monel metal rather than the graphs for ordinary steels and duralumin. The Type 5 graphs for stainless iron are accelerated-damage graphs. Under stressless corrosion, the rate of damage for stainless iron is even lower than for monel metal. For this reason, the stressless-corrosion graph for stainless iron has not yet been determined. It is probably an accelerated-damage graph.

Constant-Damage Diagrams:

The Type 9 graph for stainless iron, as shown in Fig. 16, is similar in form to the Type 9 graph for monel metal. The Type 9 graph for stainless iron, however, makes a much smaller angle with the axis of abscissas than does the Type 9 graph for monel metal. As might be expected, the direction of the Type 9 graph for stainless iron is not very different from the direction of the Type 10 constant net-damage graph for ordinary steels.

The slope of the Type 6 (*a*) graphs as shown in Fig. 17, is less for stainless iron than for monel metal.

Constant Net-Damage Diagrams:

As the stressless-corrosion graph for stainless iron has not yet been determined, constant net-damage graphs cannot yet be drawn. The Type 10 graphs, however, would evidently resemble the nearly straight portion of the Type 9 graphs, and the Type 11 (*a*) series would resemble the Type 6 (*a*) series. The constant net-damage surface for stainless iron, therefore, is probably a nearly plane surface with very slight slope. The greatest slope is probably in a direction nearly parallel to the axis of cycles. Net damage, for stainless iron as for ordinary steels, depends chiefly on corrosion stress and number of cycles and depends very little on corrosion time.

Comparison of Three-Dimensional Diagrams

Comparison of the three-dimensional diagrams for ordinary steels, duralumin, monel metal and stainless iron can be made only tentatively at present. Comparison of the five Type 11 (*a*) series shown in Fig. 22, however, indicates that the distance of the constant net-damage surface from the origin of coordinates depends chiefly on the corrosion resistance of the metal. The slope of the constant net-damage surface, however, evidently depends on something in addition to corrosion resistance. For steels the slope of this surface is greater than for duralumin, but less than for monel metal. It seems possible that the slope depends somewhat on the endurance limit of the metal. This subject, however, needs further investigation.

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Note to Figures

In Type 5 diagrams, the phrase "at this point" means that the fatigue limit, estimated from a two-stage experiment, is as indicated in the diagram. The phrase "above this point" means that for some reason the fatigue limit could not be estimated definitely from this two-stage experiment, but is estimated to be above the value indicated in the diagram.

The arrow tips used in some of the diagrams indicate by their direction: either that a specimen failed in the first stage of a two-stage experiment; or that, in order to reduce the resultant fatigue limit to the chosen value, the corrosion time must be greater than the value indicated by the position of the arrow tip.

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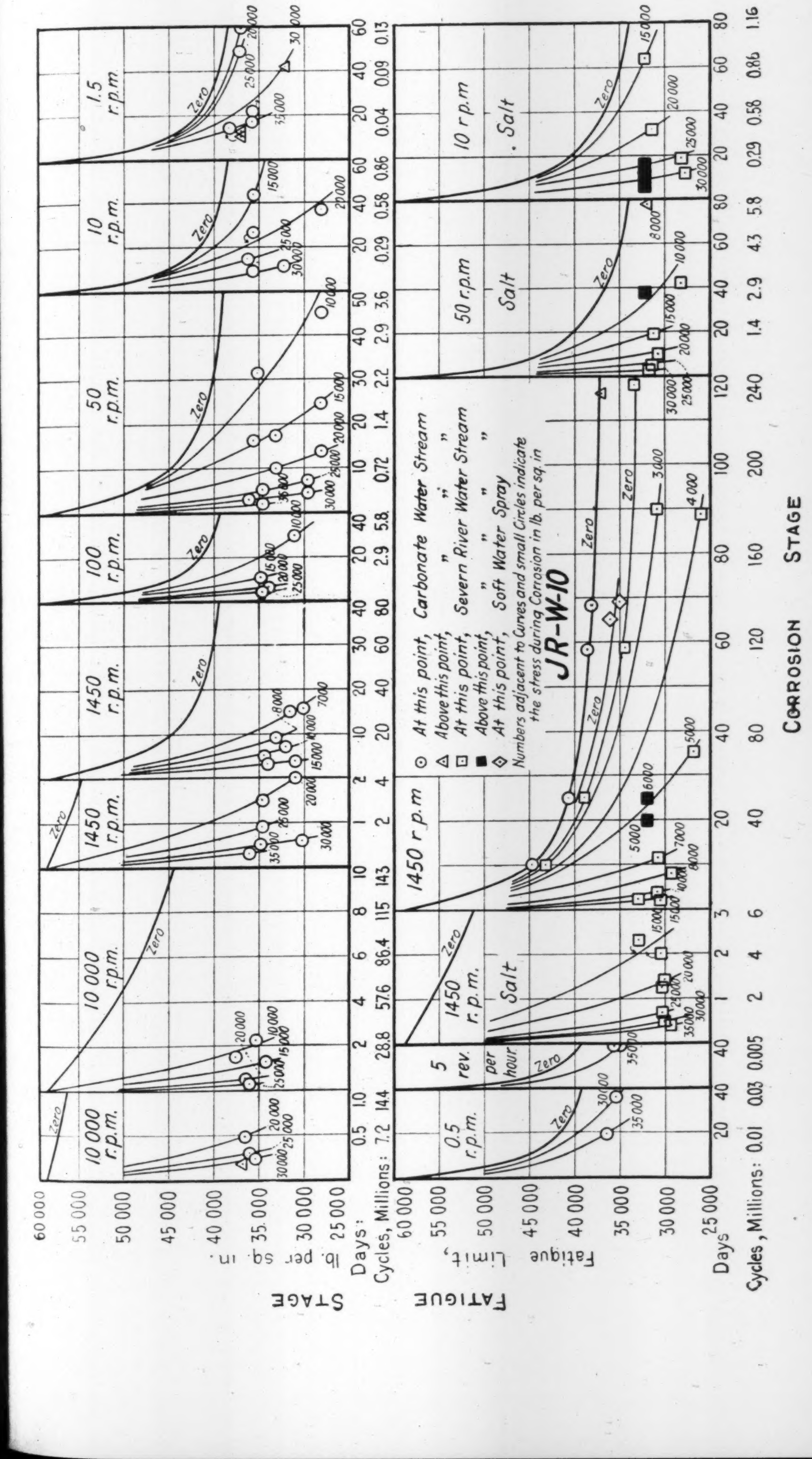

FIG. 1.—Type 5 Diagrams, $S\frac{N}{T}$. FT, Carbon Steel.

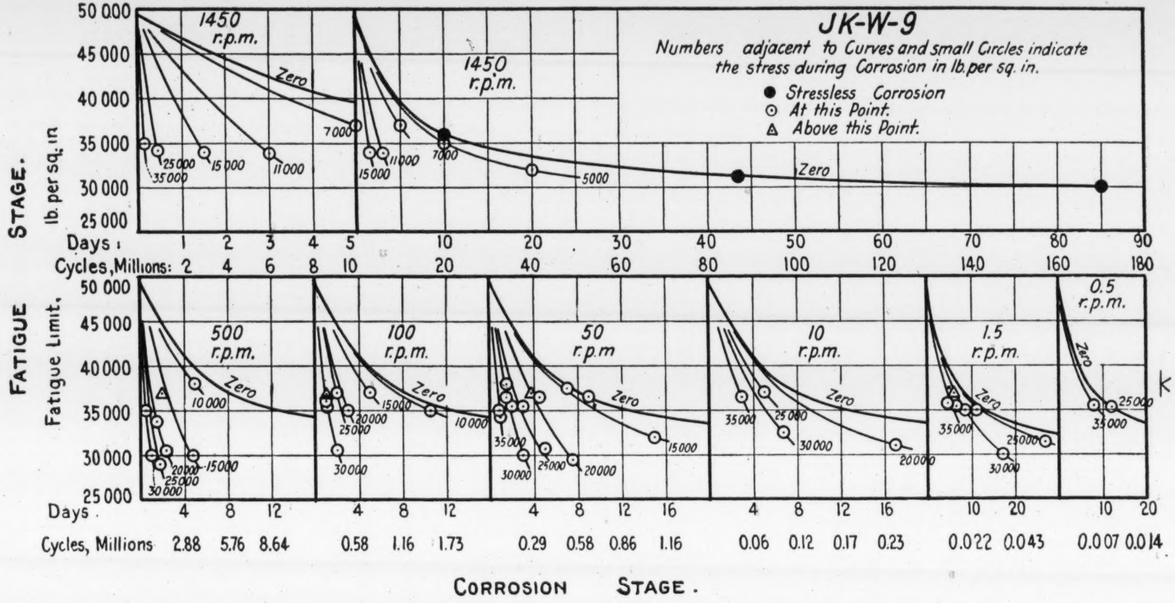

FIG. 2.—Type 5 Diagrams, $S\frac{N}{T}\cdot FT$, Copper Steel.

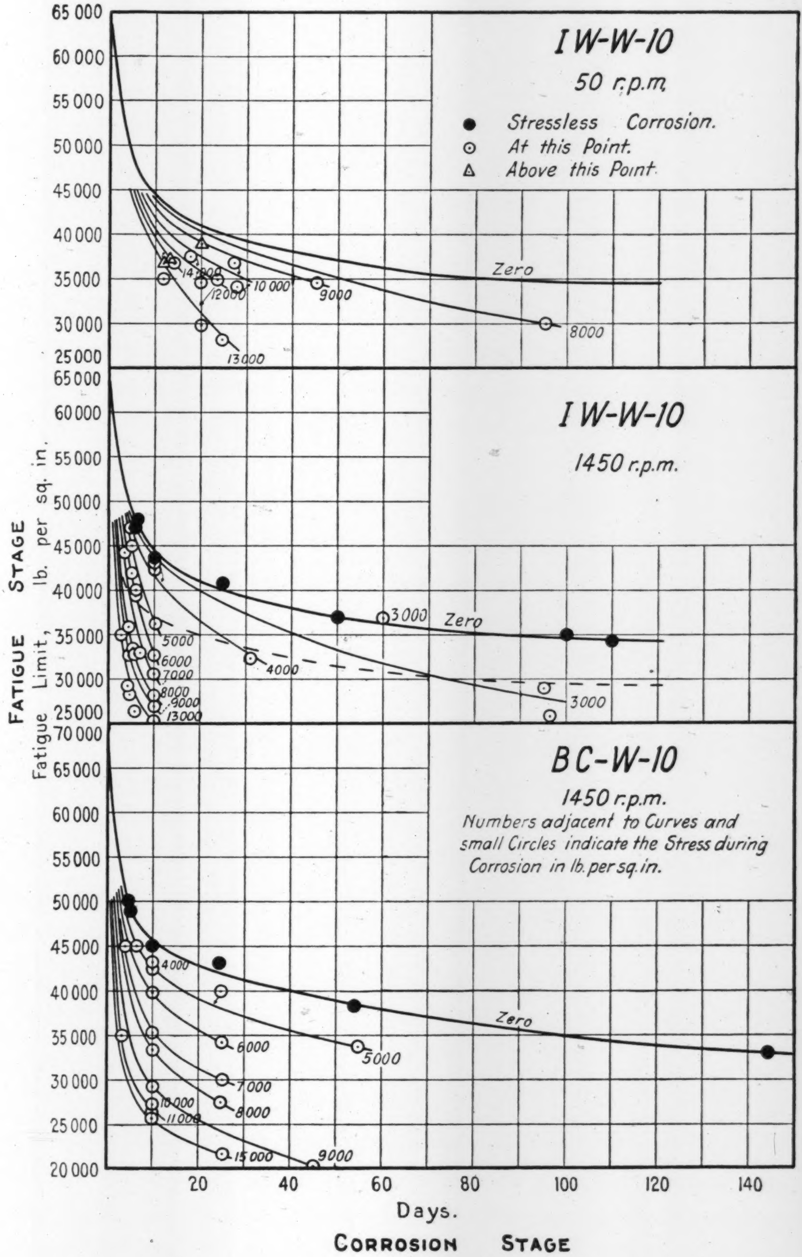

Fig. 3.—Type 5 Diagrams, $S\frac{N}{T}.\ FT$, Nickel Steel and Chromium-Nickel Steel.

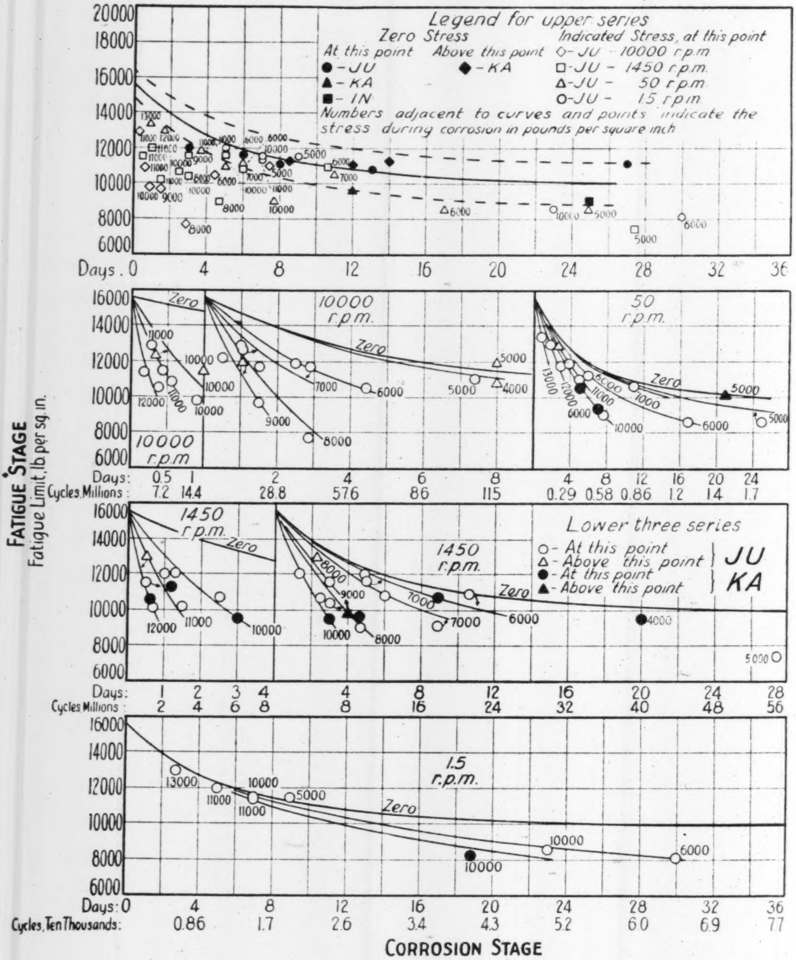

FIG. 4.—Type 5 Diagrams, $S\frac{N}{T}.FT$, Duralumin.

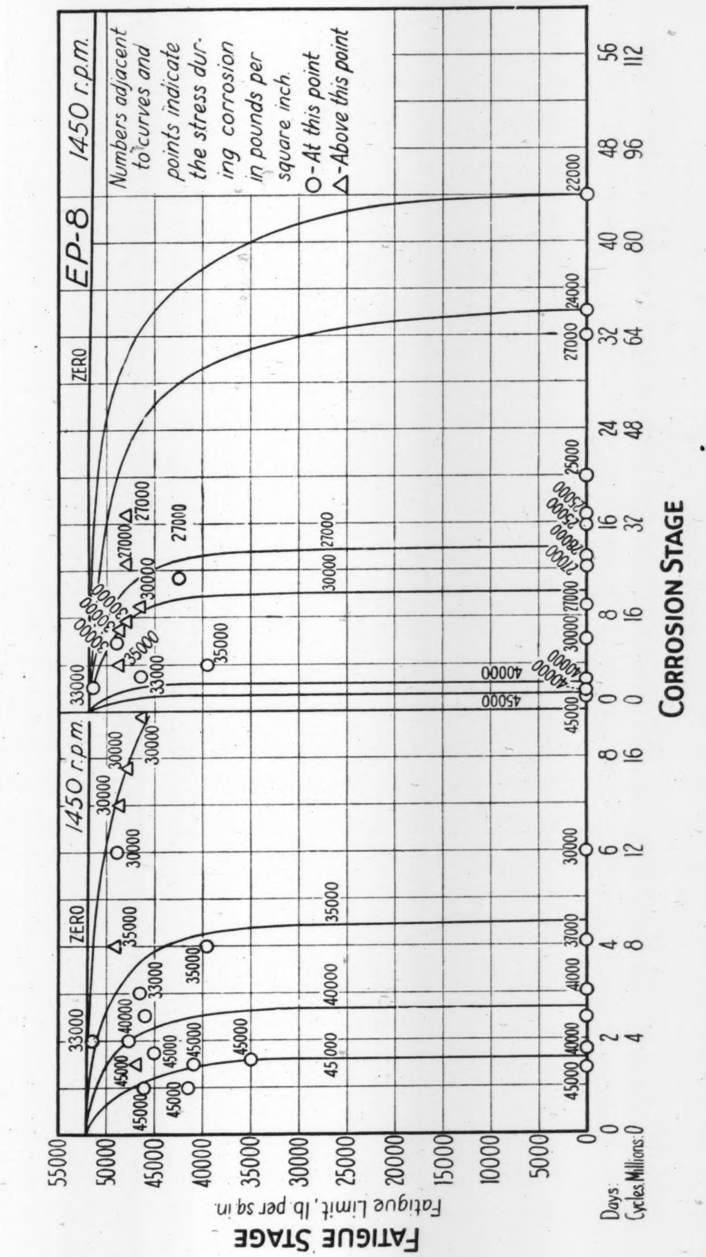

FIG. 5.—Type 5 Diagrams, $S\frac{N}{T}$, FT, Monel Metal.

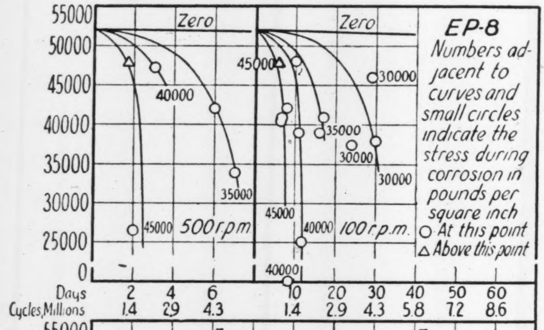

FIG. 6.—Type 5 Diagrams, $S\frac{N}{T} \cdot FT$, Monel Metal.

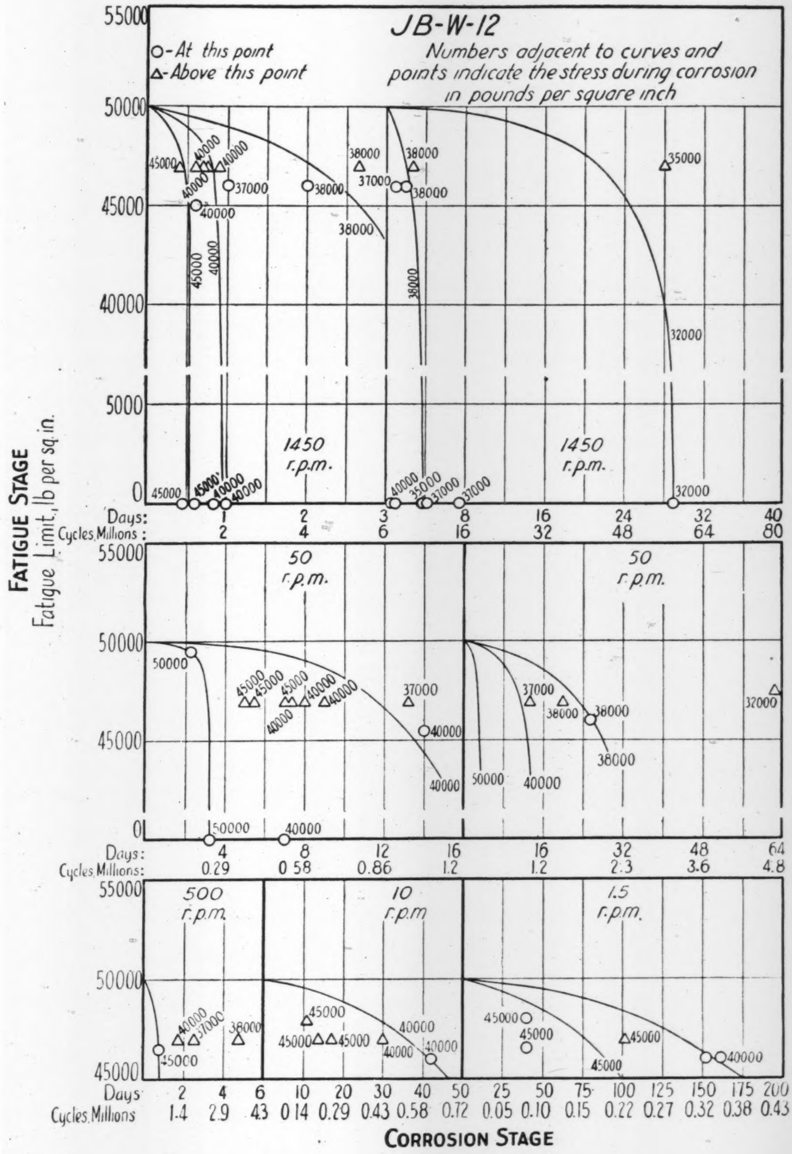

FIG. 7.—Type 5 Diagrams, $S\frac{N}{T} \cdot FT$, Stainless Iron.

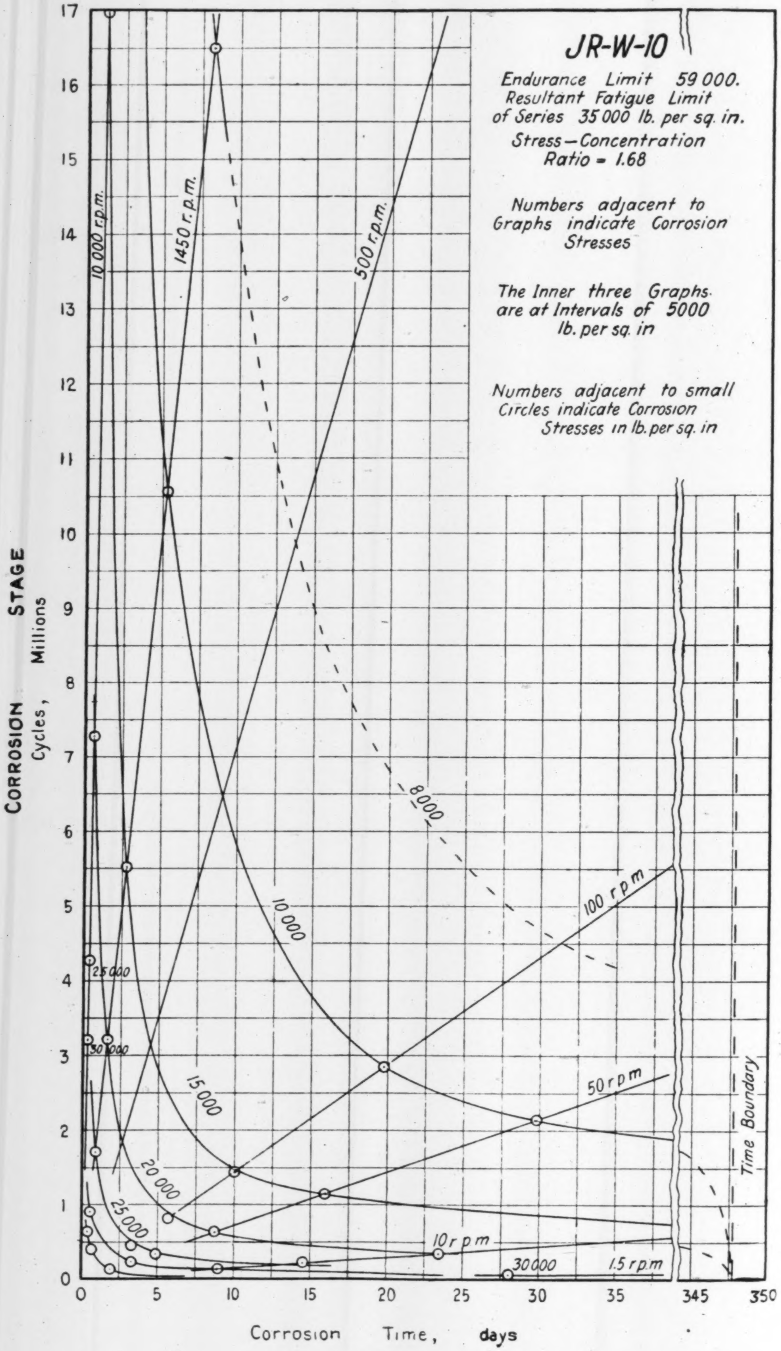

FIG. 8.—Type 9 Constant-Damage Series, *FS.NT*, Carbon Steel.

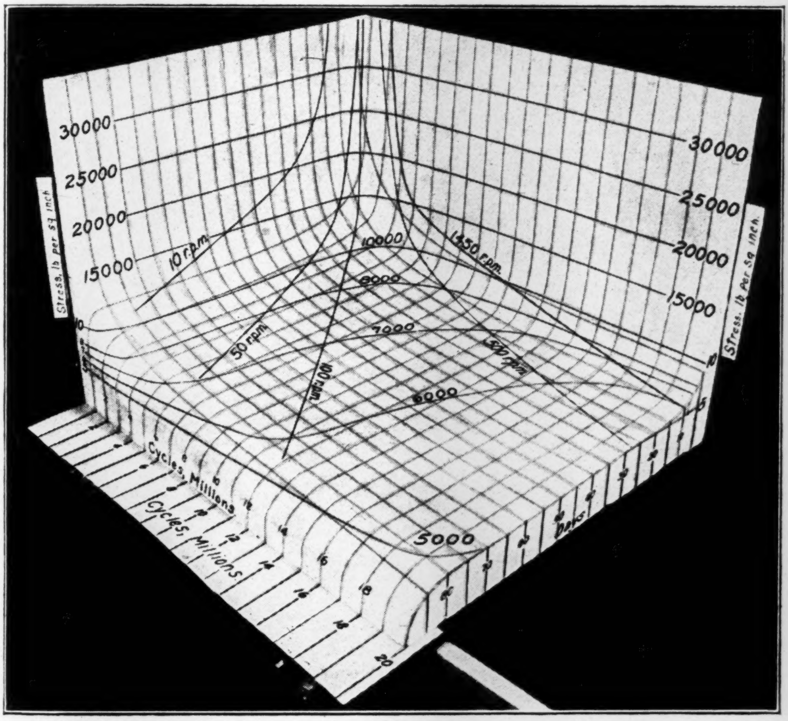

Fig. 9.—Three Dimensional Diagram, Nickel Steel, IW-W-10.

Endurance limit, 64,000 lb. per sq. in. Resultant fatigue limit, 35,000 lb. per sq. in. Stress-concentration ratio, 1.83.

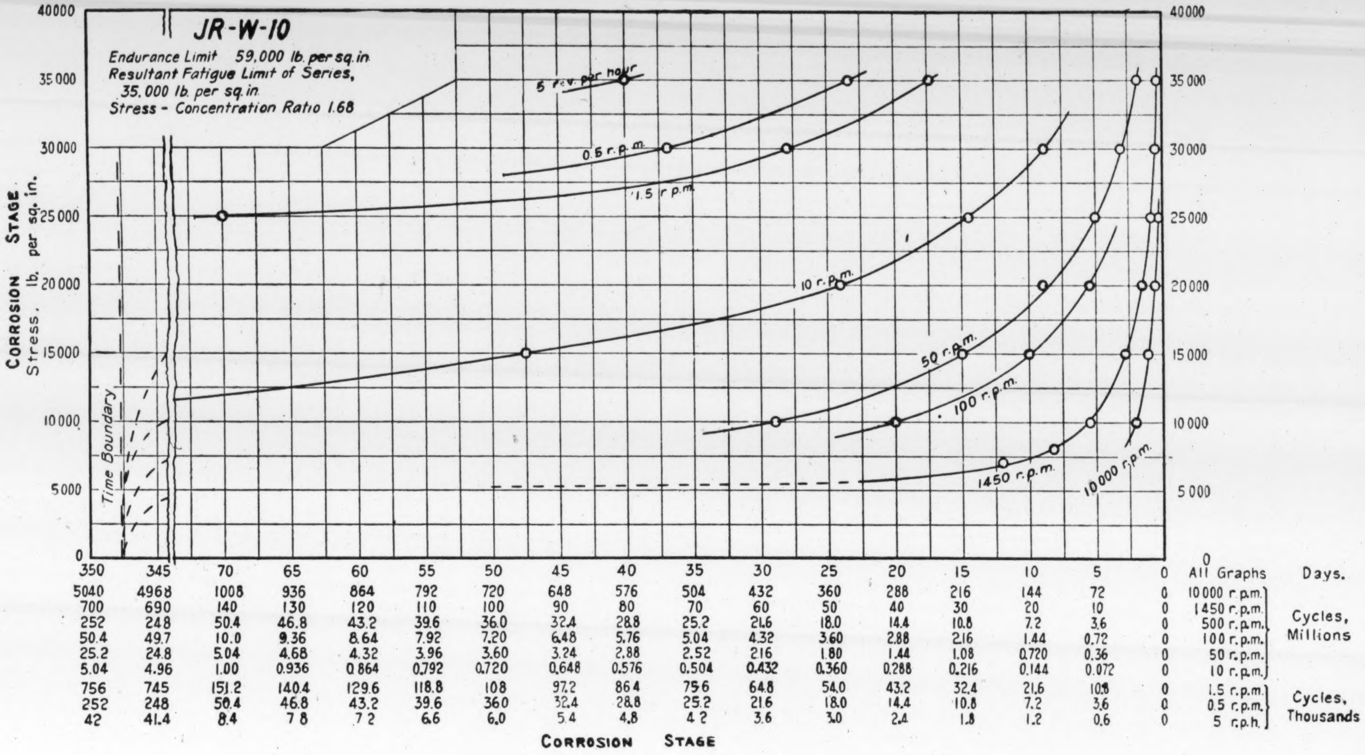

Fig. 10.—Type 6 (a) Constant-Damage Series, $F\frac{N}{T} \cdot ST$, Carbon Steel.

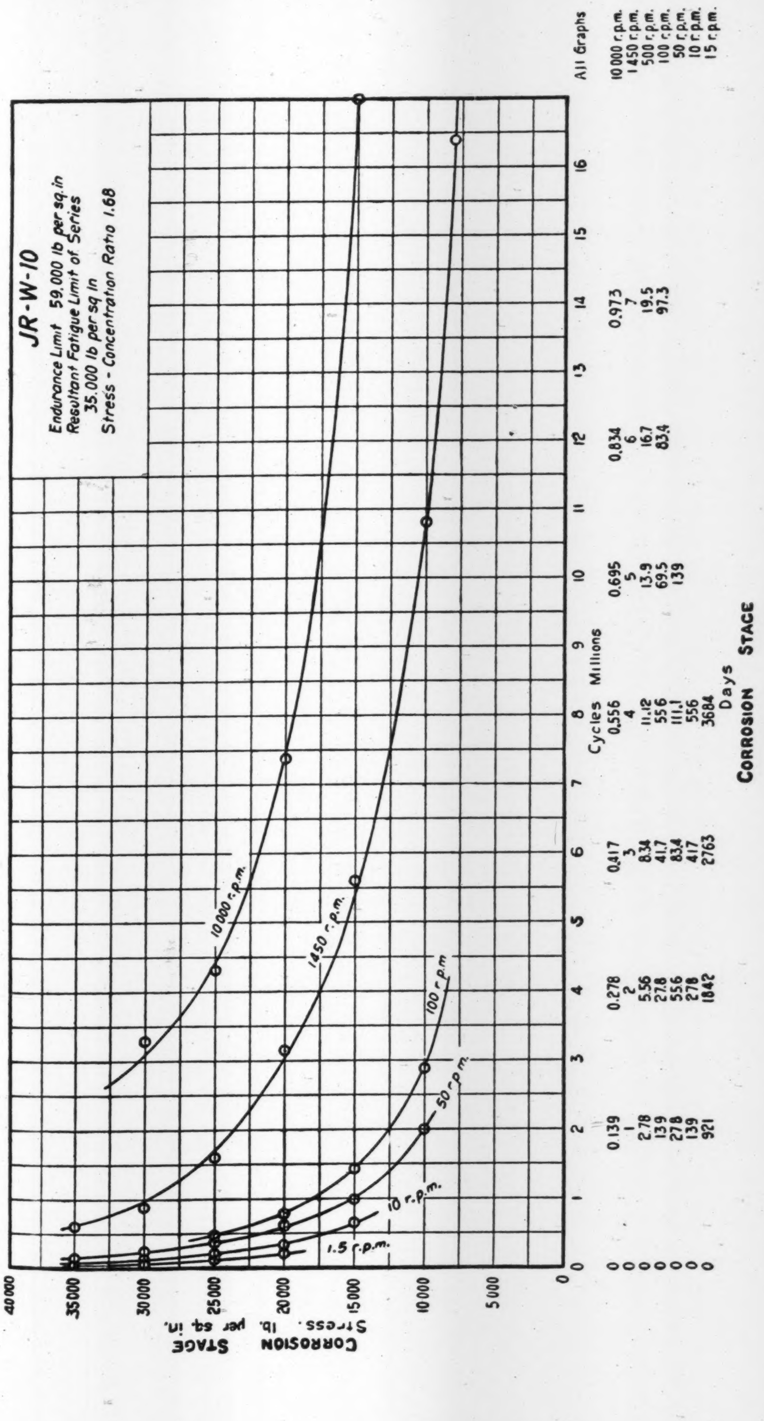

FIG. 11.—Type 6 (*b*) Constant-Damage Series, $F\frac{N}{T}$, *SN*, Carbon Steel.

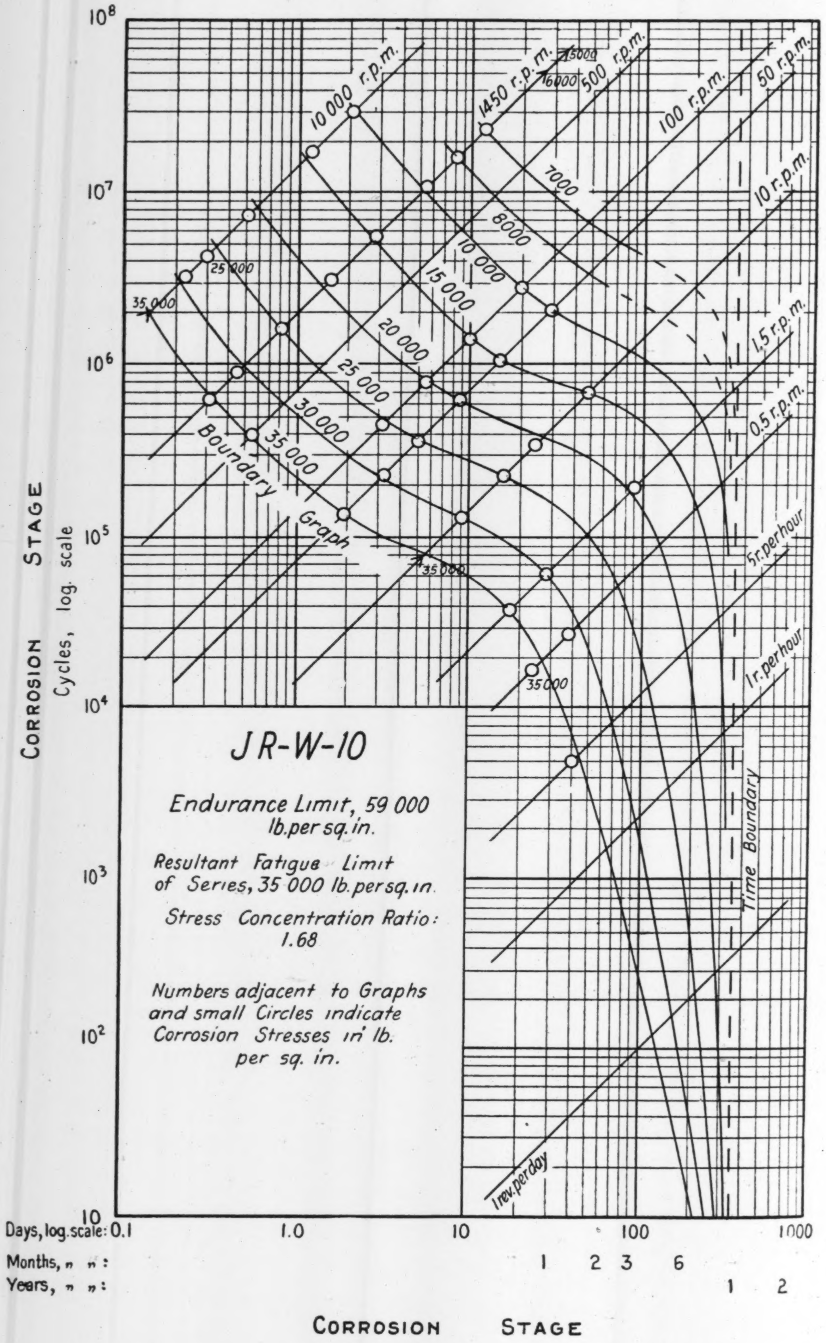

Fig. 12.—Type 9 Constant-Damage Series, *FS.NT*, Carbon Steel.

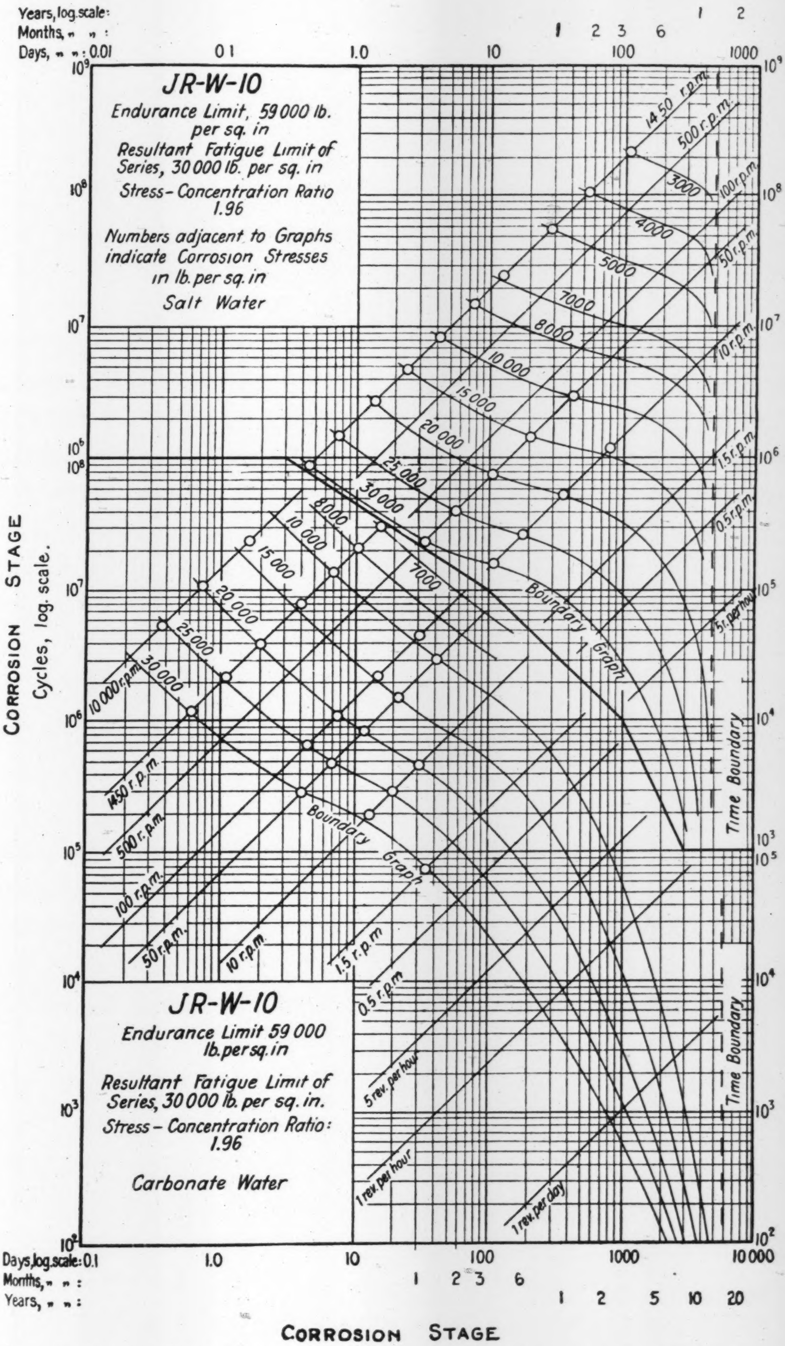

FIG. 13.—Type 9 Constant-Damage Series, *FS.NT*, Carbon Steel.

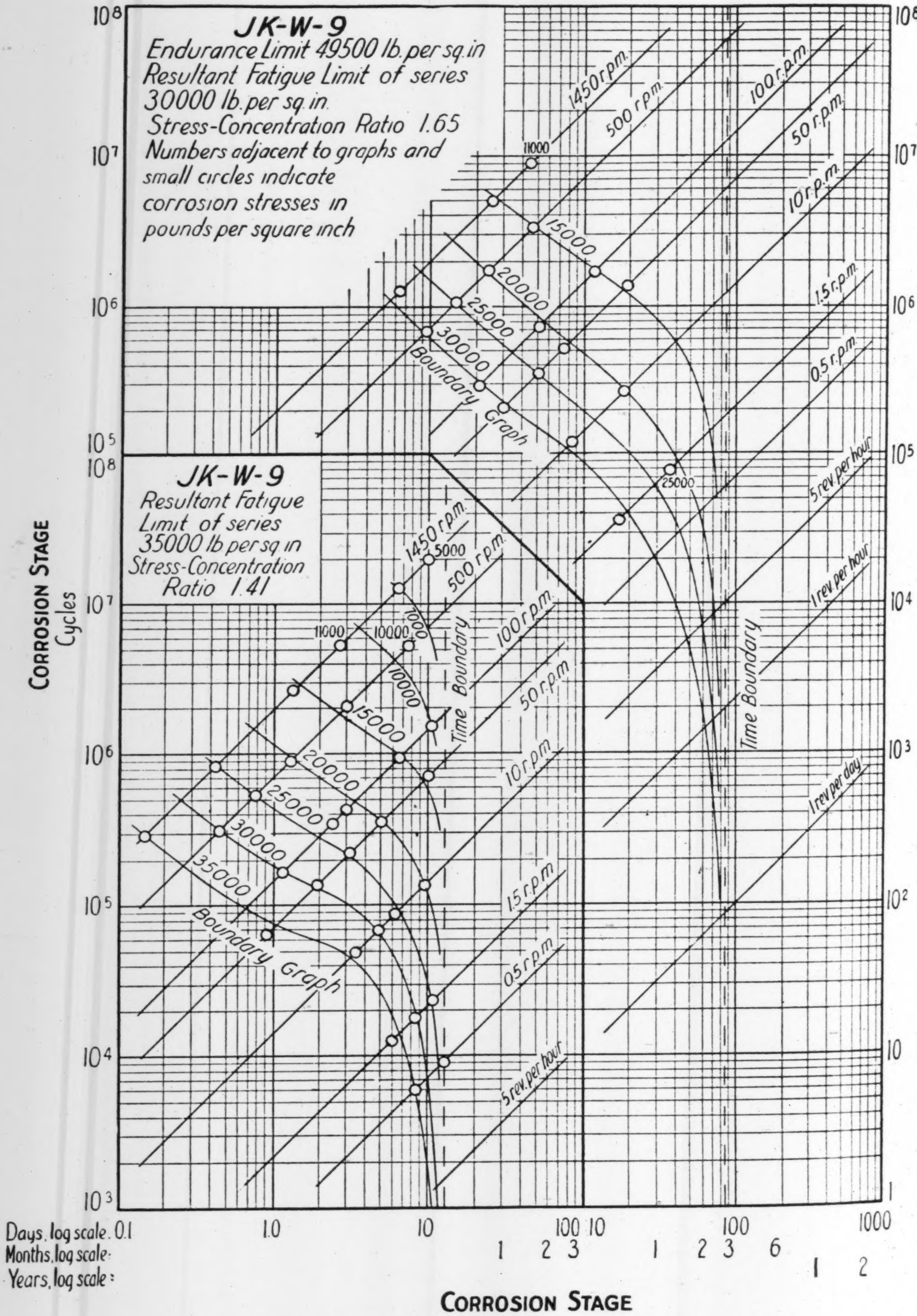

Fig. 14.—Type 9 Constant-Damage Series, *FS.NT*, Copper Steel.

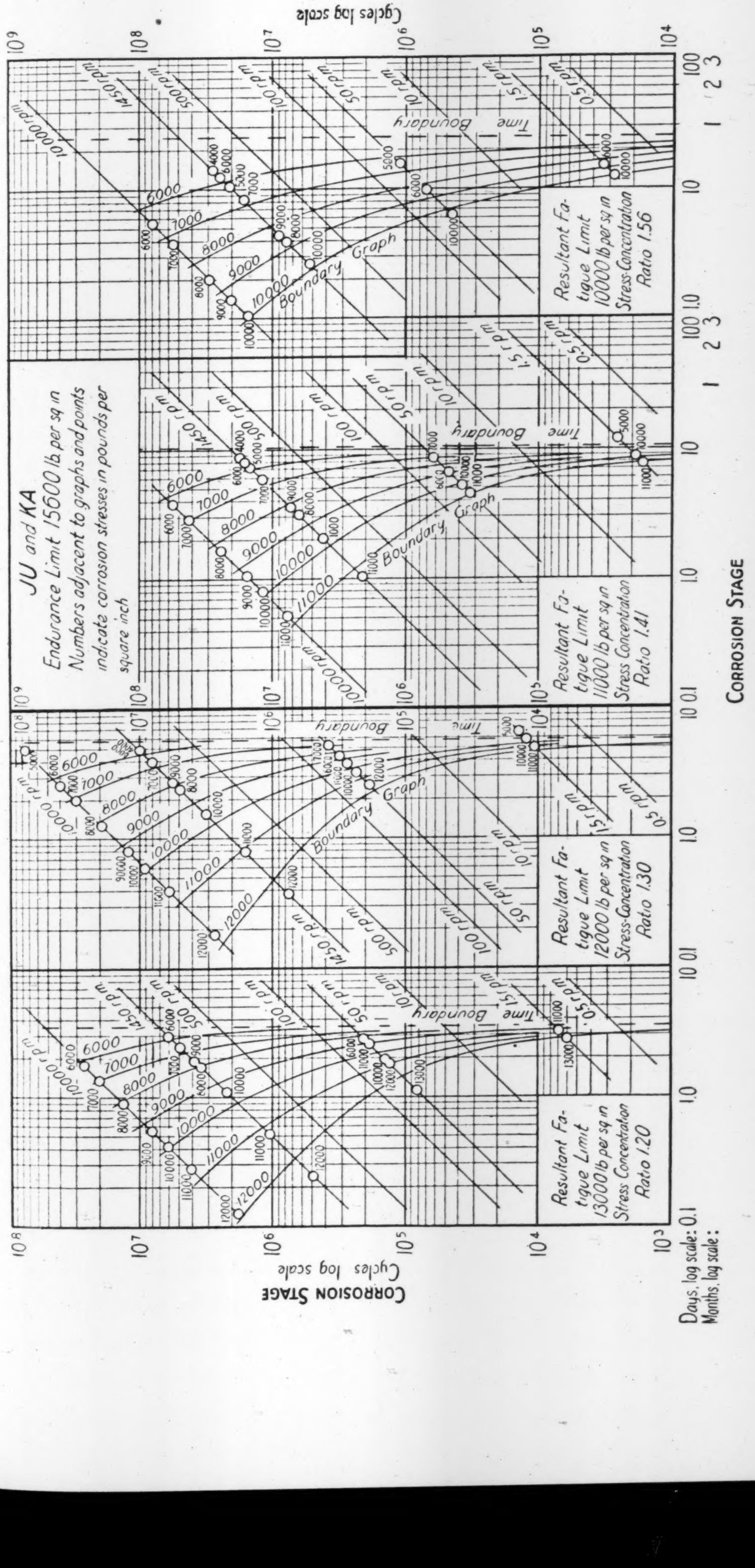

FIG. 15.—Type 9 Constant-Damage Series, FS.NT, Duralumin.

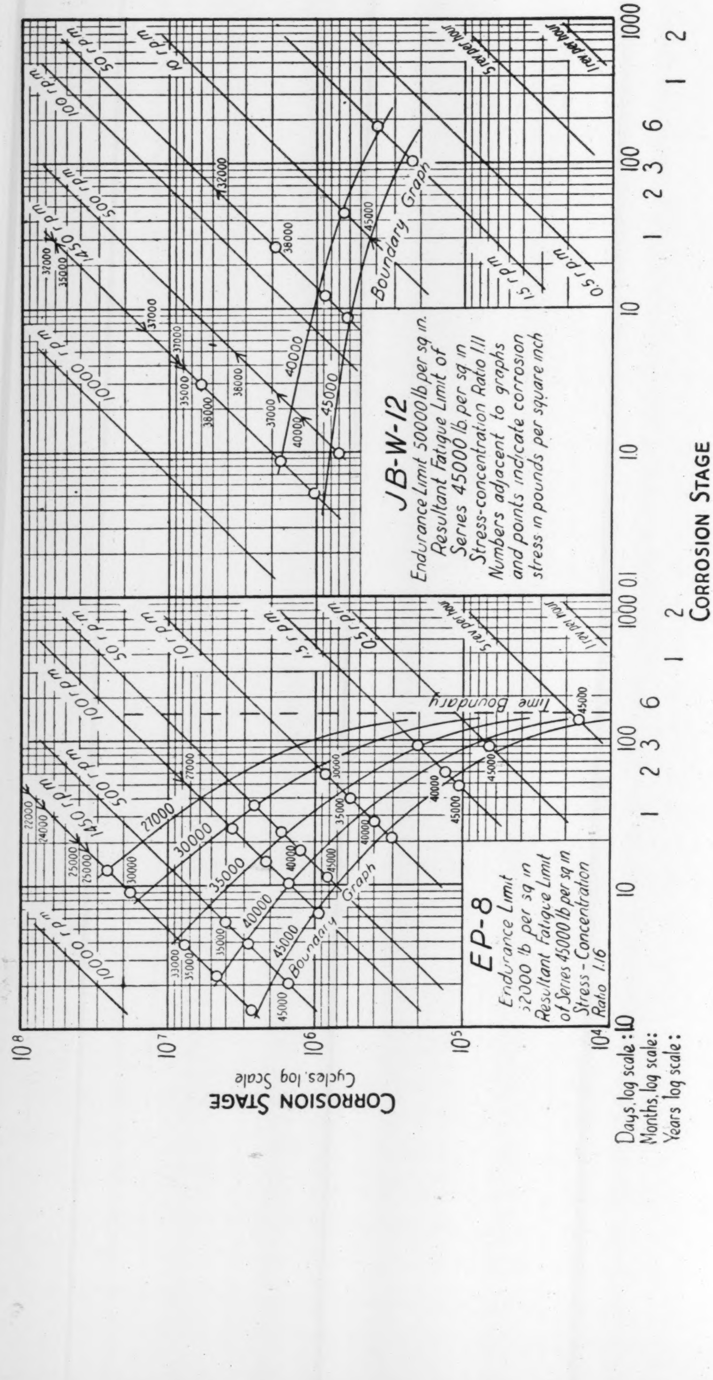

FIG. 16.—Type 9 Constant-Damage Series, *FS.NT*, Monel Metal and Stainless Iron.

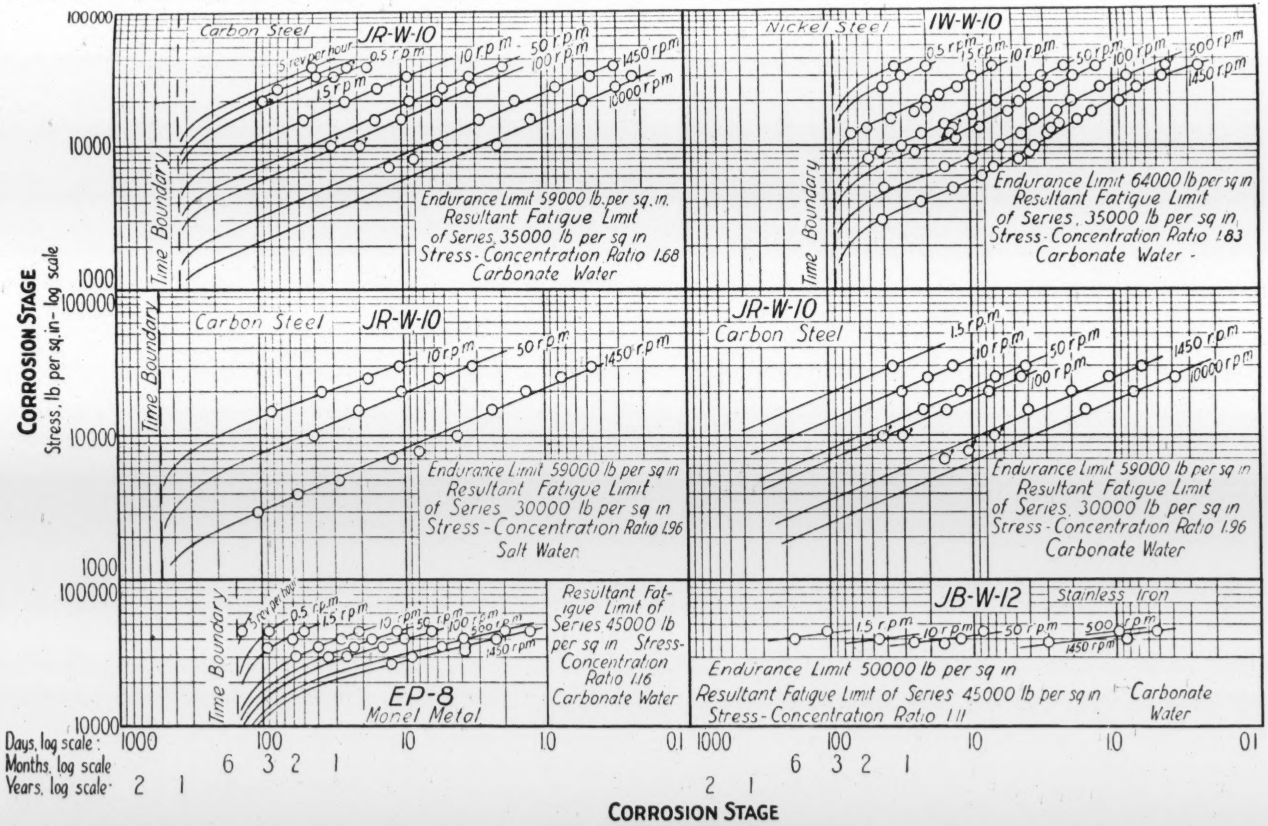

FIG. 17.—Type 6 (*a*) Constant-Damage Series, $F\frac{N}{T}\cdot ST$, Carbon Steel, Nickel Steel, Monel Metal and Stainless Iron.

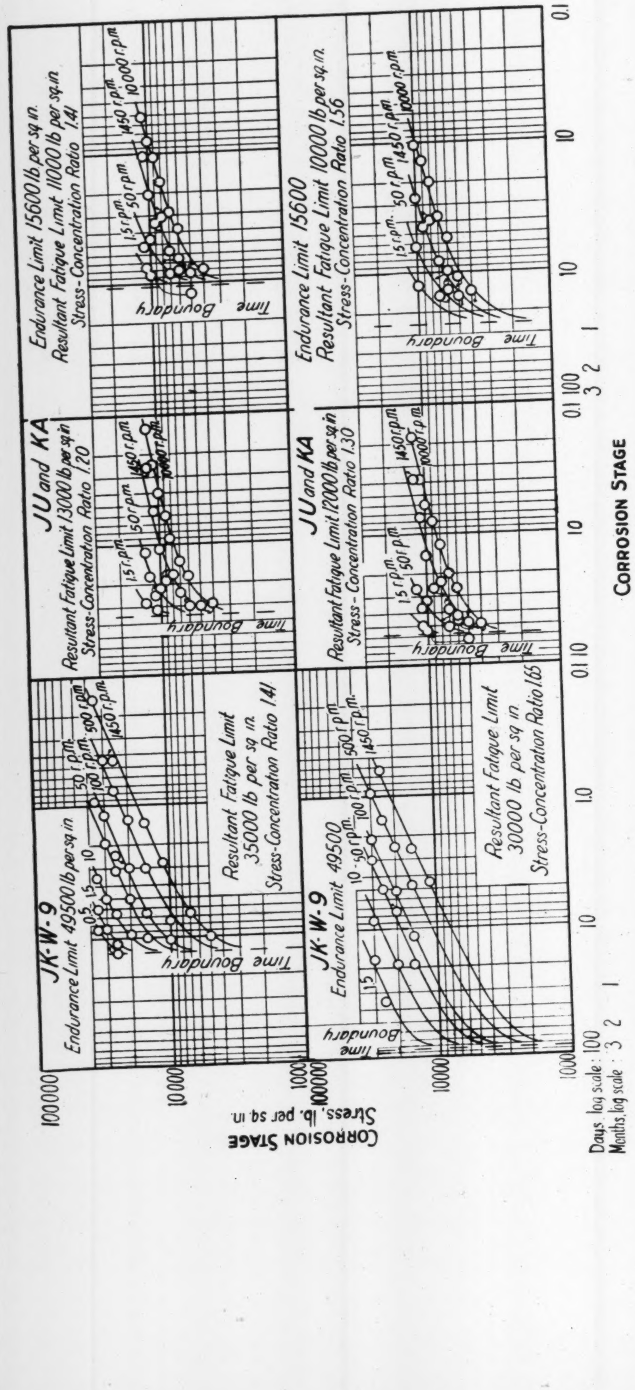

Fig. 18.—Type 6 (*a*) Constant-Damage Series, $F\frac{N}{T}\cdot ST$, Copper Steel and Duralumin.

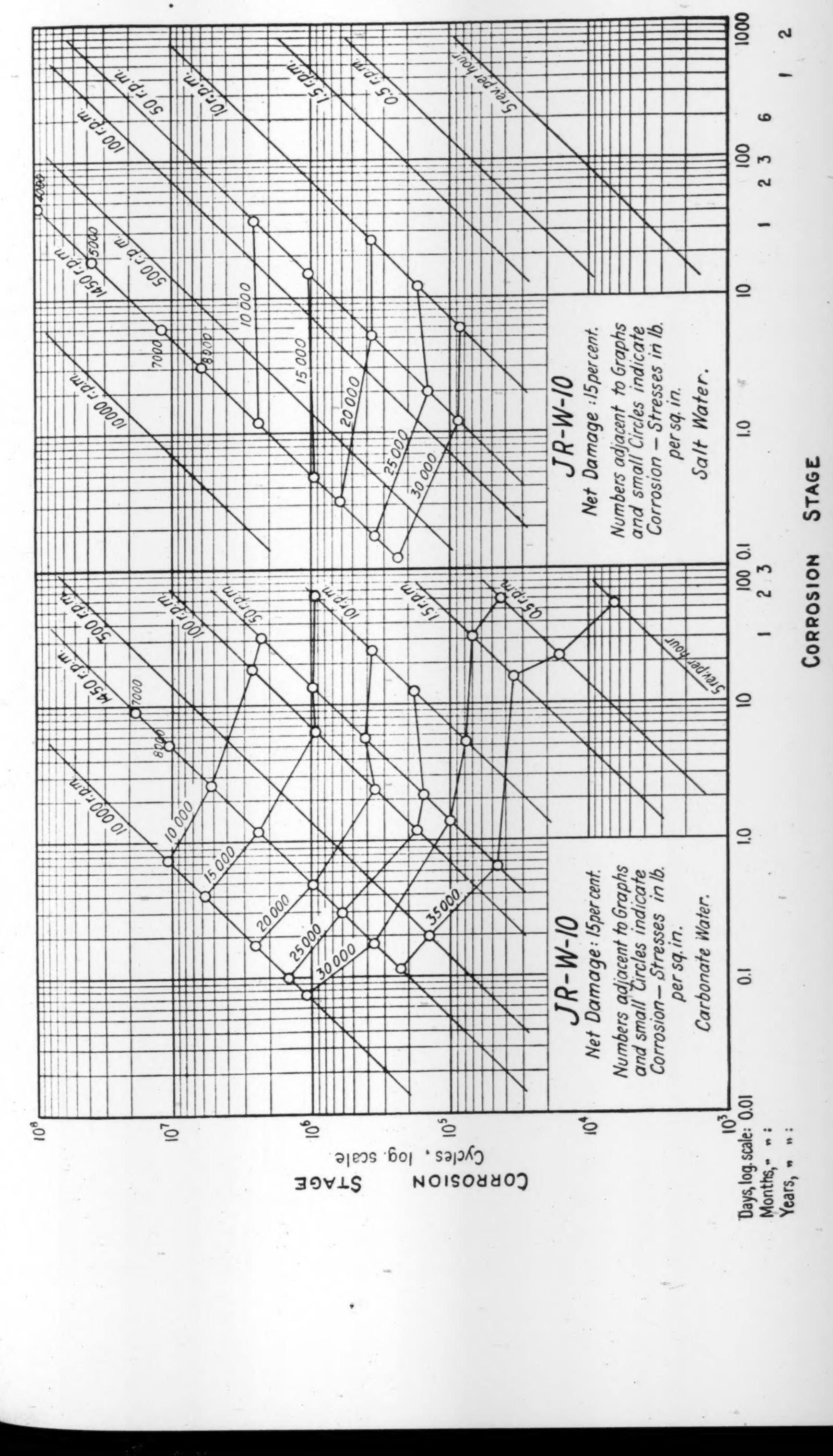

Fig. 19.—Type 10 Constant Net-Damage Series, *DS.NT*, Carbon Steel.

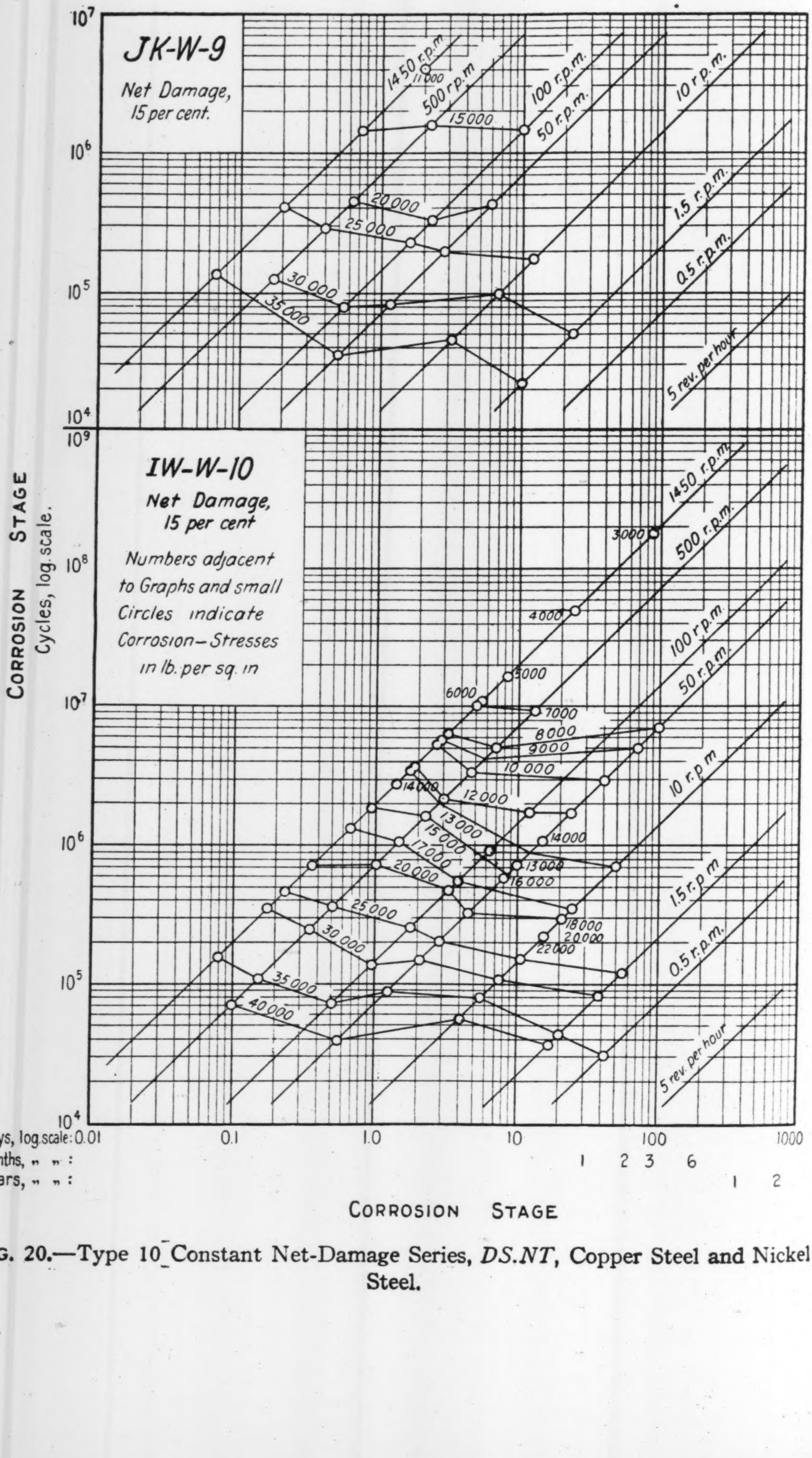

FIG. 20.—Type 10 Constant Net-Damage Series, *DS.NT*, Copper Steel and Nickel Steel.

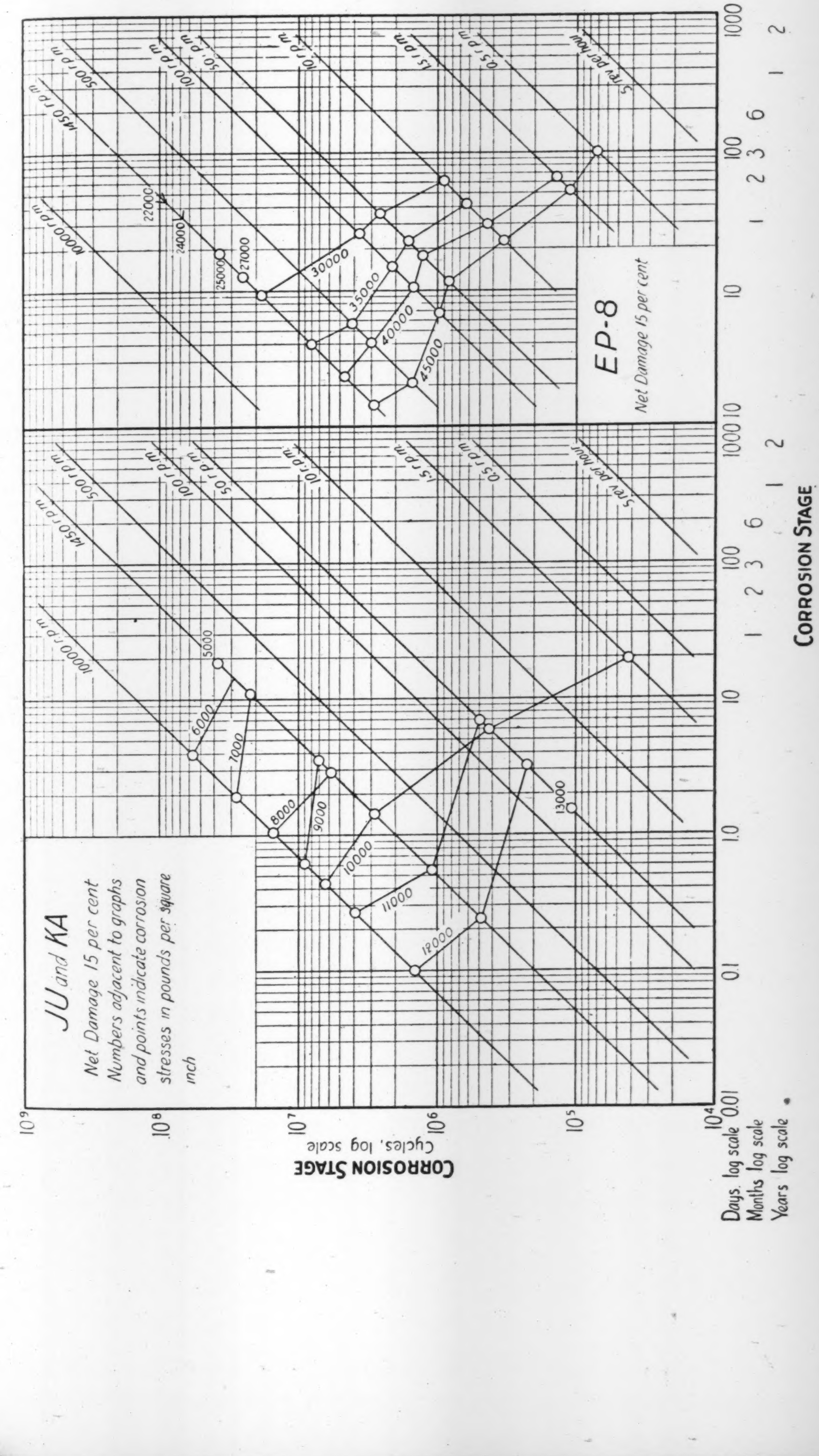

Fig. 21.—Type 10 Constant Net-Damage Series, *DS.NT*, Duralumin and Monel Metal.

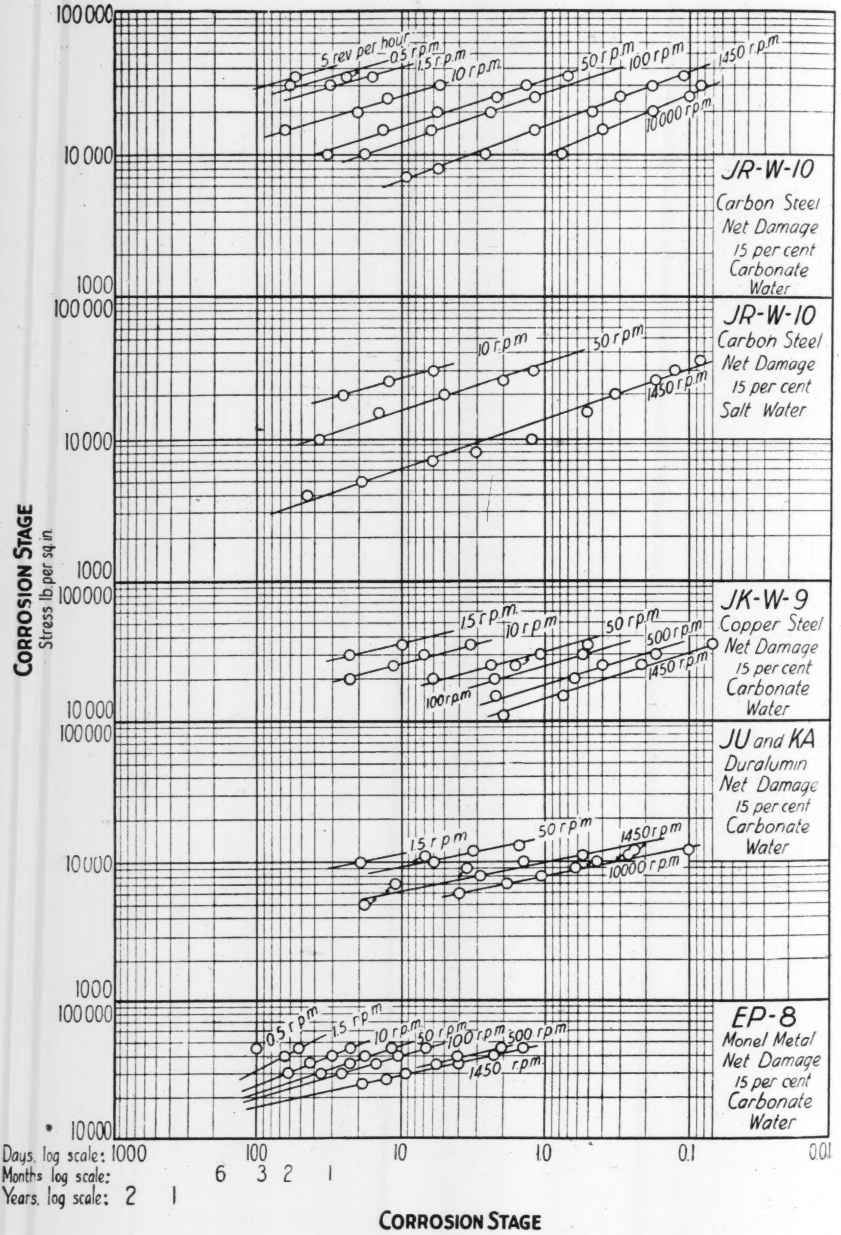

Fig. 22.—Type 11 (*a*) Constant Net-Damage Series, $D\frac{N}{T}\cdot ST$, Carbon Steel, Copper Steel, Duralumin, and Monel Metal.

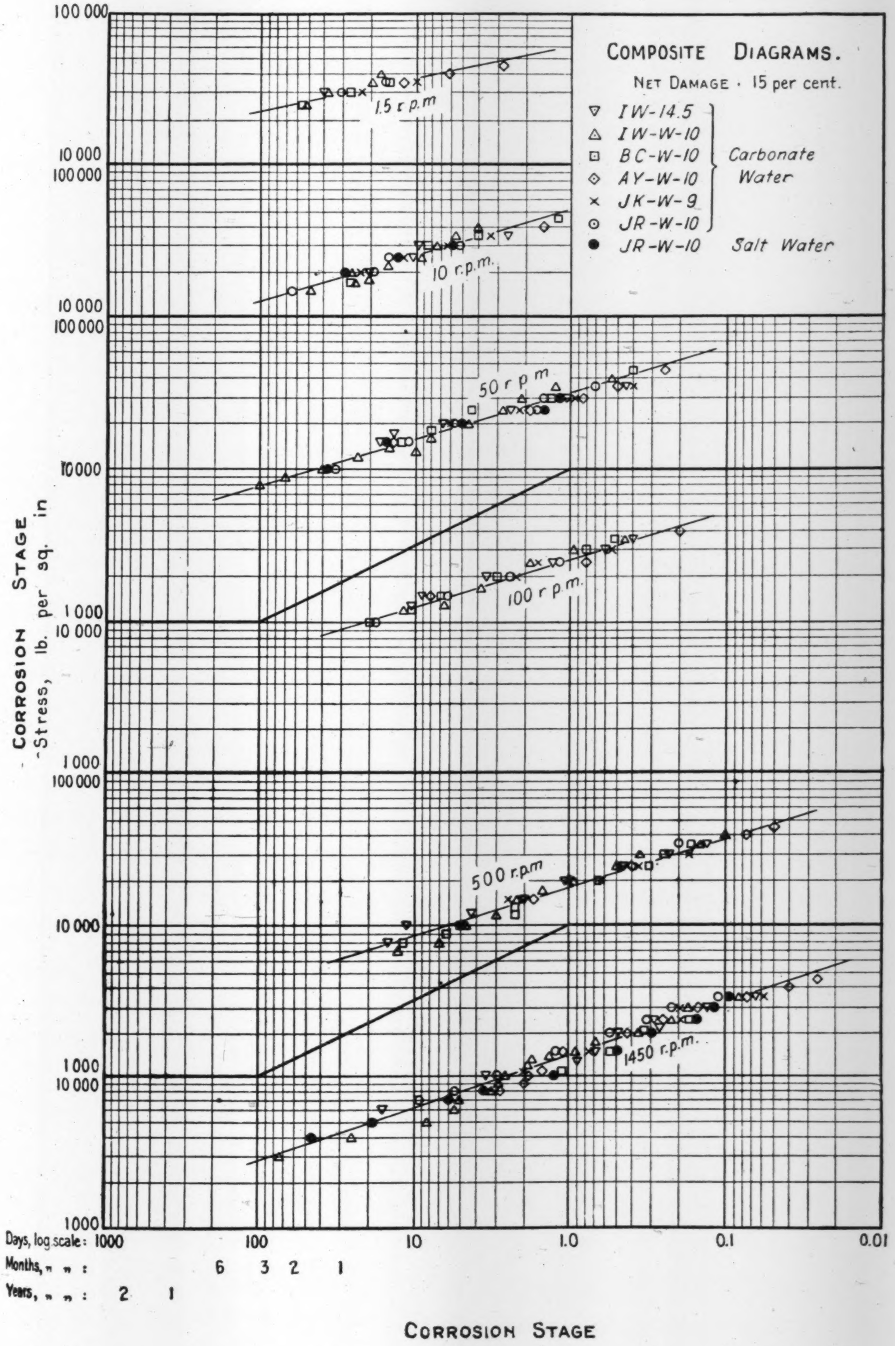

Fig. 23.—Composite Diagrams, Constant Net-Damage, Type 11 (*a*), $D\frac{N}{T}.ST$, Ordinary Steels.

DISCUSSION ON CORROSION-FATIGUE OF METALS

Mr. R. L. Templin.[1]—I wish to comment on one fact brought out in Mr. McAdam's paper. The scatter of the data in the case of duralumin would appear to be partially explained by the results obtained by Mr. Kommers on cast iron wherein the scatter of the points was found to be caused by the test specimens not running true. I should like to ask Mr. McAdam how much consideration he gave to this factor in his work.

In our fatigue testing work in the laboratories of the Aluminum Co. of America we give considerable attention to this point and our test specimens are made to run true within 0.001 in. before the test conditions are considered satisfactory.

Mr. D. J. McAdam, Jr.[2]—The amount of scatter obtained with duralumin is of merely incidental interest in connection with the paper. The "scatter" of results obtained with duralumin cannot be attributed, as Mr. Templin implies, to vibration of the test specimen. While we do not always use a micrometer in lining up specimens, investigation has convinced me that the alignment of our specimens is of the order of accuracy mentioned by Mr. Templin as desirable.

If, as Mr. Kommers suggests, cast iron is more sensitive than steel to inaccuracy of alignment, this is probably due to the large percentage of non-metallic volume in cast iron. In duralumin there are no such discontinuities as those in cast iron.

Moreover the aluminum-manganese alloy 3SH, and other aluminum alloys, show much less "scatter" than does duralumin. It seems evident, therefore, that the wide "scatter" found in some samples of duralumin is due to the nature of the material.

The published results of other laboratories also show wide scatter obtained with most samples of duralumin. Mr. Templin has not yet presented stress-cycle graphs obtained in his laboratory. It would be of interest if he would present such graphs.

Mr. F. N. Speller.[3]—For a long time stress has been recognized as a factor affecting the rate of corrosion; therefore, it seems that a clearer conception of the influence of cyclic stress on corrosion can be obtained by this view rather than by considering corrosion as affecting the mechanical properties of the metal, the viewpoint emphasized by Mr. McAdam.

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[1] Chief Engineer of Tests, Aluminum Co. of America, New Kensington, Pa.

[2] Metallurgist, U. S. Naval Engineering Experiment Station, Annapolis, Md.

[3] Director, Department of Metallurgy and Research, National Tube Co., Pittsburgh, Pa.

As the pehnomena essentially involve the formation of a nick at the most stressed section by local corrosion, followed by fatigue of the metal at the reduced section, all the factors causing localized corrosion or pitting should be considered in connection with corrosion-fatigue. These include stress, concentration cell effects, and breaks in the metal surface film, all tending to increase the potential of the metal at the most strained part. A clearer view of the mechanism of this type of corrosion may perhaps be obtained by considering what has been learned recently about surface films on metal exposed to air and other corroding agencies. The recent work of Ulick R. Evans and others has shown how important a part oxide films play in corrosion. As the oxides are cathodic to the metal, the breaking of the film by stress starts the damage that results in the destruction of the metal by fatigue. The stress at the center of the test specimen being greater than that in the adjoining metal causes the most strained part to be more anodic. When the pit becomes deep enough the oxygen concentration tends to become less at the bottom of the pit and thus still further increases the corroding rate. All these tendencies work together to keep the corrosion localized at the most-strained area.

It will be seen that the stress acts in two distinct ways, and the pitting effect of any applied stress will vary with the character of the surface film which in turn varies with changes in the corroding media. Thus while corrosion-fatigue reduced to its bare essentials may be considered as the influence of a sharp pit acting like a mechanical nick in lowering the endurance limit under stress, the many factors involved make this one of the most complicated of corrosion problems.

It would be interesting to obtain the corrosion-fatigue curve of the same metal with a wider range of composition of water and to know more about the variations in the waters used in McAdam's experiments. The difference in corrosion-fatigue in the brackish and carbonate water shown by McAdam is very significant. Stressless immersion tests in these would probably give relatively very different results as indicated by the results of A.S.T.M. immersion tests in Severn River water and other waters.[1]

We have shown in our paper[2] that the fatigue limit is lowered by increasing the chloride concentration.

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[1] See Report of Sub-Committee V on Total Immersion, Committee A-5 on Corrosion of Iron and Steel, ***Proceedings***, Am. Soc. Testing Mats., Vol. 29, Part I. p. 143 (1929).

[2] F. N. Speller, I. B. McCorkle and P. F. Mumma, "The Influence of Corrosion Accelerators and Inhibitors on Fatigue of Ferrous Metals," see p. 238.

From the work already done, it is obvious that without waiting for a more complete mathematical analysis, corrosion preventives should be applied wherever there is danger of stress and corrosion acting together in metal structures.

I should like to ask whether Mr. McAdam made comparative tests with pure water, tap water and highly saline waters. Even with an oiled surface there would be some corrosion. In order to get a true endurance-limit curve without any corrosion or thermal effect, would it not be advisable to recirculate pure water carrying a sufficient amount of chromate in solution to entirely inhibit corrosion. Such a water will maintain a thin protective film on the metal even under rapidly alternating stresses, so that tests made under these conditions should show the true fatigue limit.

MR. MCADAM.—In regard to the first question by Mr. Speller, we used soft water and also Severn River water. The soft water was artesian well water with very low saline content. The Severn River water has about one-sixth the salt content of sea water.

The thermal effect mentioned by Mr. Speller was discussed by the author in a previous paper.[1] Except in special cases, changes in the fatigue limit due to this thermal effect are so slight as to be of merely academic interest. For the materials used in the investigation the extra trouble involved in the use of a sodium chromate solution would not be justified. Fatigue limits obtained in air are sufficiently accurate.

MR. H. F. MOORE.[2]—A statement of results of a few tests run at the University of Illinois, while by no means giving a conclusive answer to the questions raised by Mr. Speller, yet may be of interest. Fatigue tests on an alloy steel were run subjecting some specimens to a stream of tap water—the same kind of water which has caused caustic embrittlement in boiler plate—and subjecting other specimens to a jet of wet steam. The wet steam would not be pure water by any means but would be nearer pure water than the tap water. The reduction in endurance limit from the value for tests in air was about one-half as great for the wet steam as for the tap water.

A recent letter from Parker Haigh of the British Royal Naval Academy at Greenwich states that for certain metals, especially lead, the fatigue strength of the metal in air may be appreciably increased by coating the metal with a thick layer of grease, or by testing it in a bath of oil. The steels did not show much increase of fatigue strength due to this protection against air corrosion.

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[1] D. J. McAdam, Jr., "Stress-Strain-Cycle Relationship and Corrosion-Fatigue of Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 224 (1926).

[2] Professor of Engineering Materials, University of Illinois, Urbana, Ill.

MR. McADAM.—In determining corrosion-fatigue limits in wet steam, attention should be given to the amount of oxygen present. The higher corrosion-fatigue limits reported by Mr. Moore are probably due to the fact that less oxygen is present in the steam than in ordinary tap water.

MR. U. R. EVANS[1] (*by letter*).—Mr. McAdam deserves the thanks of all engineers for his detailed survey of the vitally important subject of corrosion-fatigue; such a study of the effect of all the different variables is absolutely necessary if we are to interpret the phenomenon.

The question is raised in the paper whether the low oxygen concentration in the pits may be an important factor in corrosion under cyclic stress, and Mr. McAdam quotes my own work on pitting under stressless conditions. The principle of differential aëration undoubtedly contributes to the initiation of pitting; but I agree with Mr. McAdam that in most cases it is mainly the effect of accumulating corrosion products which determines the rate at which the fatigue strength deteriorates. Indeed last year,[2] in discussing the plugging of pits with corrosion product in stressless corrosion, I ventured to suggest that the conception of "plugging" might help also to interpret Mr. McAdam's results on corrosion-fatigue.

I will restate the case as follows. If no external stress is applied, corrosion starts at certain points where (either owing to surface defects, or inclusions, or internal stresses) the invisible primary oxide film is weak or discontinous; such points are anodic towards the parts where the primary film is in good repair and the insoluble secondary corrosion products (such as rust) shield these points from oxygen, which might otherwise repair the primary film; thus anodic attack continues at these points, burrowing down into the metal and producing pits; the further down the pit burrows, the better is oxygen excluded, and at first sight the action should become quicker as it burrows deeper. This acceleration in the rate of destruction is actually realized—as Mr. McAdam's curves show it. Monel metal and stainless steel, materials which readily build a very obstructive primary film, and where consequently even a trace of oxygen (such as may enter the pits in the initial stages) would interfere perceptibly with the rate of deepening. But in ordinary steel and duralumin, another factor comes in. The rapidly formed secondary corrosion products (which must be clearly distinguished from the primary oxide film) occupy a bigger volume than the metal destroyed, and therefore congestion is likely to occur in the pits tending either (1) to disrupt the metal,

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[1] Cambridge, England.

[2] *Journal*, Soc. Chemical Industry, Vol. 47, p. 55 T. (1928).

(2) to force out the corrosion product, or (3) to clog the pit and bring the burrowing to a standstill.

As to which of these three things will occur will depend on the strength of the metal, the plasticity of the secondary corrosion product, and the expansion involved in the corrosion. But it is quite clear that if the first prevails, the weakening will continue at continually increasing speed, and if the third prevails it will diminish with the time. Thus it is not surprising to find an accelerated weakening with some materials and a retarded weakening with others.

If now we superimpose alternating stresses, clearly we aid the disruption, assist the expulsion and diminish the clogging. Hence metals like steel, in which the rate of weakening falls off considerably with the time in the so-called "stressless" corrosion, show more persistent weakening if cyclic stresses are applied. Qualitatively this idea explains most of the results, but when we note the extraordinary effects of increasing stress and frequency, it appears likely that the alternations must be doing something more than what has been suggested above. The clogging and development of pressure by secondary products is a slow process, as Mr. McAdam's stressless curves show clearly; it seems somewhat unlikely that an increase of frequency could make so marked a difference in the rate of weakening, unless the function of the cyclic stress is also to prevent some other more rapid process which would diminish weakening in stressless corrosion. Actually we know of such a process, namely, the partial repair of the primary film by the small amount of oxygen which will penetrate even into pits; if the alternating stresses are fast enough they will break this skin more rapidly than it can be repaired, and since the metal at the anodic part is then maintained in a completely "active" condition, the solution pressure, as Mr. McAdam words it, will be "not the solution pressure of stressless corrosion but an enhanced solution pressure due to the cyclic stress." This is probably the main reason for the rapid weakening at high frequencies and high stresses.

The fact that the building up of the primary film is a process enormously quicker than the clogging with secondary corrosion product is proved by independent researches coming from three different centers. Fruendlich's[1] observations at Berlin indicate that a very short exposure of oxide-free iron even to dry air builds up a layer of oxide which, although invisible to the eye, can be detected optically; the same conclusion was reached independently by myself at Cambridge[2] (England), by a different method, whilst McAulay's potentiometric work at Hobart[1] (Tasmania) points in a similar direction. Finally the particularly elegant work of Speller, McCorkle and Mumma[2] at Pittsburgh show that chromates, which are known to keep the film in repair in stressless corrosion, prevent the setting in of corrosion-fatigue, whilst chlorides, which militate against their action in stressless corrosion, also do so when the corrosion is accompanied by cyclic stresses. There appears little doubt that damage to the primary film—as well as opposition to clogging by secondary products—must be considered when interpreting the action of alternating stress corrosion.

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[1] *Zeitschrift Physicalische Chsmie*, Vol. 128, p. 321; Vol. 130, p. 289 (1927).

[2] *Journal*, Chemical Soc., p. 1020 (1927).

MR. H. E. SMITH.[3]—Referring to the paper by Speller, McCorkle and Mumma, I should like to ask if in using rubber rings, there was any indication of special corrosion due to the sulfur in the rubber? Some years ago we attempted some simple corrosion experiments, not in connection with fatigue at all. We had our specimens on glass rods with rubber separators, and found streaks of corrosion down from the separators. In petroleum works, the corrosion by the sulfur in crude petroleum in connection with water in the tanks is very pronounced.

MR. SPELLER.—Of course there is a possibility of sulfur compounds playing a part, but in the short time the tests described in our paper were going on, we found that materials other than rubber would give the pitting effect; so this effect is evidently due to the shielding of part of the surface from oxygen, and not so much to the material in contact with the metal.

MR. EVANS (*by letter*).—The measurements recorded in the paper by Speller, McCorkle and Mumma are of special value because they indicate the extent to which corrosion-fatigue trouble can be met by suitable chemical treatment of the water involved; moreover they contribute greatly to our understanding of this hitherto baffling phenomenon.

The explanations suggested appear sound, and are the more acceptable in that they tally well with those suggested by recent work on the corrosion of metal specimens which are unstressed—or carry only residual stresses. There seems no doubt that the invisible oxide film normally present on ordinary iron or steel keeps cracking or breaking down, and we only get immunity from attack when there is present at all points an oxidizing agent in sufficient quantity to repair the film as quickly as repair is needed. If the supply at some place is insufficient for proper repair, then anodic attack will commence at that place, and the presence of the oxidizing agent at the places where the film is in good repair actually stimulates corrosion at the points of breakdown, since it acts as a cathodic depolarizer. Thus the same body may either act as inhibitor or stimulator of corrosion, according to its concentration and distribution. Dissolved oxygen can act in both ways. It stimulates corrosion at the places where its concentration is low, and paradoxically prevents it where the concentration is high. In the case of pure iron in pure water, we get corrosion if the oxygen concentration is not uniform; but the same iron in the same water is passive and unattacked if the oxygen concentration is maintained uniform at all points, by means of a simple apparatus described elsewhere.[1] Stainless iron or steel has a film more easily repaired by oxygen than that of pure iron. With ordinary commercial iron or steel it is impractical to attempt to avoid corrosion in this way; but by using another oxidizing agent, potassium chromate, inhibition often becomes a practical proposition, although even here the amount of chromate required must be increased if chlorides are present since these ions penetrate weak places where other ions cannot pass.

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[1] *Journal*, Chemical Soc., p. 85 (1929).

[2] F. N. Speller, I. B. McCorkle and P. F. Mumma, "The Influence of Corrosion Accelerators and Inhibitors on Fatigue of Ferrous Metals," see p. 238.

[3] Engineer of Materials, New York Central Lines, New York City.

Alternating stresses keep cracking the film continually and thus accelerate corrosion, which in turn produces stress concentration, and thus the two factors (alternating stress and corrosion) cause more damage when they act together than when they act separately. The presence of a good film repairer, for example, potassium chromate, or the use of a material, for example, stainless steel, which builds an oxide film capable of being kept in repair by dissolved oxygen alone, offer promise of solving some of the problems; but in both cases, the matter is rendered more difficult by the presence of chlorides. It should be noticed that not only alternating stresses, but also residual stresses left by cutting or bending, produce a tendency towards film breakdown; some of my recent potential measurements[2] show that on specimens of steel sheet the film is weakest near the sheared edges, and that corrosion usually starts at these weak points and spreads outwards and downwards.

MESSRS. F. N. SPELLER AND I. B. MCCORKLE[3] (*authors' closure by letter*).—Mr. Evans, in discussion of Mr. McAdam's paper, states that "if the alternating stresses are fast enough they will break this skin more rapidly than it can be repaired." Then, conversely, it would seem that an inhibitor which did not give complete protection at a certain speed might do so at a slower speed, or at least there would be a tendency towards this protection. The results of recent tests show that speeds between 189 and 1790 r.p.m. have no effect on the film repairing power of an inhibitor (specifically, sodium dichromate). The results of these tests, on which this statement is made, are briefly summarized in the following table. The stress for all tests was 34,000 lb. per sq. in., which is 2000 lb. per sq. in. below the air endurance limit of the steel used.

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[1] "Corrosion of Metals," 1926 edition, p. 108, McGraw-Hill Book Co., New York City.

[2] *Journal*, Chemical Soc., p. 92, 111 (1929).

[3] **Department of Metallurgy and Research, National Tube Co., Pittsburgh, Pa.**

TESTS MADE IN DISTILLED WATER CARRYING 2000 P.P.M. SODIUM CHLORIDE.

TEST	SPEED, R.P.M	NUMBER OF CYCLES TO FAILURE	TIME, HOURS	CYCLE RATIO	TIME RATIO
No. 1.........	1790	1,078,000	10.0	Unit	Unit
No. 2.........	558	539,100	16.1	0.50	1.60
No. 3.........	189	373,500	32.9	0.35	3.28

TESTS MADE IN DISTILLED WATER CARRYING 2000 P.P.M. SODIUM CHLORIDE PLUS 4000 P.P.M. SODIUM DICHROMATE.

TEST	SPEED, R.P.M.	NUMBER OF CYCLES TO FAILURE	TIME, HOURS	CYCLE RATIO	TIME RATIO
No. 4.........	1790	6,889,700	64	Unit	Unit
No. 5.........	558	3,635,200	109	0.53	1.69
No. 6.........	189	2,187,800	193	0.32	3.01

It will be noted that as the speed decreases in both series, the number of cycles to failure decreases, but the time increases. Also, it is evident that the inhibitor gave partial protection at all speeds. But the important point is the constancy of the ratios of time and cycles at different speeds, with and without inhibitor, even though the numerical vaules are different. If there had been a tendency for the slower speeds to aid the film building, then the values of the ratios for the inhibitor carrying solution would seemingly tend to increase to some higher value than the ratios for the solution without inhibitor. Different results might be obtained at a wider range of frequency. These results may be explained by assuming that the change in stress causes a constant disturbance and partial disintegration of the film originally formed on the surface rather than a cracking of the film at each cycle, and that this is apparently independent of the cycle frequency. More experimental work is needed before a definite explanation of the effect of frequency can be offered.

The above data, obtained by the simultaneous action of corrosion and stress to failure, are in general accord with results reported in Mr. McAdam's paper.

MR. MCADAM (*author's closure by letter*).—Mr. Speller's statement that "for a long time stress has been recognized as a factor affecting the rate of corrosion" may tend to give an incorrect impression. In discussing the effect of stress on corrosion, clear distinction should be made between the effect of plastic deformation prior to corrosion and the effect of stress simultaneous with corrosion. That previous cold working makes metal more susceptible to corrosion has been known for a long time. That cyclic stress (even stress below the elastic limit), if applied during corrosion, accelerates corrosion, has not been recognized for a long time.

In view of the fact that in three preceding papers [1, 2, 3] and in the present paper I have discussed and emphasized the influence of cyclic stress on corrosion, it is indeed surprising to read in the first paragraph of Mr. Speller's discussion, that the present paper has not been devoted to the influence of cyclic stress on corrosion, but to "considering corrosion as affecting the mechanical properties of the metal." In a still earlier paper[4] on the general phenomena of corrosion-fatigue, I expressed the conclusion that these phenomena are due to the accelerating influence of cyclic stress on corrosion. The object of the investigation described in this and three preceding papers, [1, 2, 3] therefore, was to obtain quantitative information on this subject.

In these four papers the fatigue limits of previously corroded specimens were presented and discussed. I have tried to emphasize, however, by the titles of the papers and by repeated statements in the papers, that these fatigue limits were determined primarily for use as criteria or measures of the "damage" due to corrosion. The chief object was to measure the effects of cyclic stress on corrosion. The effects of cyclic stress on corrosion have been illustrated quantitatively in the paper by numerous diagrams.

Although this was the primary object, the information obtained on the effects of corrosion on "mechanical properties" is of great practical importance. Space limitation, however, has prevented a discussion of this subject in the paper.

In reply to the fourth paragraph of Mr. Speller's discussion I would refer him to Fig. 1, which illustrates among other things the effect of stressless corrosion in carbonate water and in Severn River water.

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[1] D. J. McAdam, Jr., "Corrosion of Metals as Affected by Time and by Cyclic Stress," *Technical Publication No. 58*, Class E, Inst. of Metals, No. 21, Am. Inst. Mining and Metallurgical Engrs. Issued with *Mining and Metallurgy*, February, 1928.

[2] D. J. McAdam, Jr., "Some Factors Involved in Corrosion and Corrosion-Fatigue of Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 117 (1928).

[3] D. J. McAdam, Jr., "Corrosion of Metals as Affected by Stress, Time and Number of Cycles," *Technical Publication No. 175*, Class E, Inst. of Metals, No. 62, Am. Inst. Mining and Metallurgical Engrs. Issued with *Mining and Metallurgy*, February, 1929.

[4] D. J. McAdam, Jr., "Fatigue and Corrosion-Fatigue of Metals," paper presented at Internat. Congress Testing Mats., Amsterdam, September, 1927, and published in the Congress *Proceedings*, Vol. 1, p. 305 (1927).

Mr. Evans' clear discussion of the various factors involved in corrosion of metals under stress is very welcome. The distinction that he draws between the primary film and the secondary corrosion products must be kept clearly in mind in any attempt to arrive at an explanation of the effect of cyclic stress on corrosion. I agree with Mr. Evans that the effect of cyclic stress on secondary corrosion products probably does not account for the great effect of cycle frequency.

Mr. Evans thinks it possible that at high frequency the cyclic stress breaks the primary film more rapidly than it can be repaired. The evidence presented by Mr. Speller in opposition to Mr. Evans' idea that "breaking" of the film may be more important at high than at low cycle frequency appears to me to be inconclusive. Comparison of numbers of cycles necessary to cause failure, especially when the number of specimens broken is only six, is not a good basis for conclusions. Moreover, corrosion-fatigue tests to destruction involve two periods, a corrosion period and a fatigue period. It is only the first period that should be considered in discussing the effects of cyclic stress on corrosion. The fatigue period in the experiments cited by Mr. Speller would probably require nearly the same number of cycles in each test. Subtracting the same number from each of his numbers (of cycles to failure) would considerably change the ratios.

Nevertheless it seems to me that the evidence now available is not inconsistent with the idea that at low as well as at high cycle frequencies the phenomena are largely due to the effect of cyclic stress on the primary film or on the metal just beneath the film. At junction of film and underlying metal there is probably some penetration of atoms of oxygen or other negative element into the space lattice of the metal. Corrosion involves diffusion of the negative atoms inward and of metal atoms outward. It seems possible that this diffusion is accelerated by distortion of the space lattice due to the cyclic stress. Stretching and compression of the film itself may assist in this acceleration. This process, it seems to me, is not usually a "breaking" of the film.

It is hoped that additional quantitative data may throw some light on this subject.

THE EFFECT OF CORROSION, ACCOMPANIED BY STRESS, ON THE TENSILE PROPERTIES OF SHEET DURALUMIN[1]

By H. S. Rawdon[2]

Synopsis

The high-strength aluminum alloy commonly known as "duralumin," an important material in aircraft construction, has been found to become quite weak and brittle occasionally while in service. As a result of a rather extensive study carried out at the U. S. Bureau of Standards in cooperation with various government agencies interested in aircraft, the reasons for this undesirable behavior have been greatly clarified.

As a part of this study, the effect upon the tensile properties of sheet duralumin of subjecting the material simultaneously to corrosive influences and to stress has received considerable attention. Full-size tension specimens were corroded in a sodium-chloride solution containing hydrogen peroxide, by an alternate-immersion method while stressed in static tension somewhat below the yield point or while being repeatedly bent back and forth in flexure. The tensile properties of the specimens after corrosion were then determined.

The effect upon the tensile properties of corrosion under these conditions is summarized in graphical form. The corrosive attack on plain duralumin sheet was found to be increased by static tension, although the increase over the attack on comparison unstressed material was not great. The attack on aluminum-coated duralumin sheet was found to be entirely negligible even with severely corrosive conditions accompanied by relatively high tensile stress.

Corrosion accompanied by repeated bending stresses was found to constitute a very searching test for corrodibility. Even here, however, the aluminum-coated duralumin is so decidedly better in its behavior than the plain duralumin as to leave no uncertainty whatever as to the superiority of this type of material for purposes where high strength, lightness and corrosion-resistance are conditions to be fulfilled.

Introduction

The effect of corrosive influences acting simultaneously with the stressing of a metal specimen by externally applied loadings, as in the determination of some of the mechanical properties of the metal, may be considered either (1) from the standpoint of the effect of corrosion on the observed mechanical properties or (2) conversely, as the effect of the acting stress in accelerating the corrosion rate of the metal. McAdam, in his pioneer work on corrosion-fatigue,[1] has emphasized the first aspect of the problem. He has shown that the observed endurance limit of a metal when it is corroded simultaneously with the stressing of the specimens is very decidedly lower than the endurance limit obtained under ordinary conditions, that is, in air. Furthermore, he has shown that the lowering of the endurance limit which results when the specimen is corroded prior to the application of fatigue stress is not so pronounced as when corrosion accompanies fatigue. Recently[2] he has discussed the other aspect of the problem, to show how the rate of corrosion is influenced by the conditions of stress which may obtain. Moore[3] has confirmed McAdam's conclusions and shown the serious effect upon the endurance limit of duralumin in the form of thin-walled tubes when corroded simultaneously with the application of cyclic stress. Speller[4] has extended this type of work and shown that by the use of a corrosion inhibitor it is possible to prevent the lowering of the "air" endurance limit when the metal is subjected simultaneously to cyclic stress and corrosive influence.

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[1] Published with the approval of the Director of the Bureau of Standards of the U. S. Department of Commerce, Washington, D. C.

[2] Senior Metallurgist, U. S. Bureau of Standards, Washington, D. C.

In numerous instances the second aspect of the stress-corrosion problem is, by far, the more important one. Removal and replacement of parts are frequently necessary solely because of deterioration resulting from corrosion. Not only is it desirable to know the corrosion behavior of materials, as determined for ordinary circumstances, but also the manner and extent to which this behavior is affected by stress or other conditions which may be imposed upon the materials in service. It is with this aspect of the corrosion problem that this paper is concerned.

It is now well recognized, by manufacturers and users alike, that sheet duralumin,[5] one of the most valuable materials of construction for aircraft, under some conditions does not maintain its initial properties without impairment. This also applies to some of the other high-strength aluminum alloys. The change, in a few cases, has been found to be very pronounced indeed and, as shown by the tensile properties, to consist in a marked lowering of the ductility accompanied by a more moderate decrease in the tensile strength. The very extensive and successful use of the high-strength aluminum alloys for aircraft and elsewhere has served in large measure to refute imputations which have been made at times concerning the general unreliability of this material. In general, experience has shown that most sheet duralumin under most conditions retains for years its initial strength and ductility unimpaired. With the increasing demands imposed upon the materials of modern aircraft construction, however, the question of permanence of such materials becomes one of increasing importance and for this reason a rather extensive investigation of the general subject of the permanence of duralumin sheet was undertaken several years ago by the U. S. Bureau of Standards in cooperation with the National Advisory Committee for Aeronautics, Bureau of Aeronautics of the Navy Department, and Army Air Corps. The manufacturers also cooperated in the work and very generously furnished the many materials needed.

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[1] D. J. McAdam, Jr., "Some Factors Involved in Corrosion and Corrosion-Fatigue of Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 117 (1928). References to the previous publications will be found in this paper.

[2] D. J. McAdam, Jr., "Corrosion of Metals as Affected by Stress, Time and Number of Cycles," *Technical Publication No. 175*, Am. Inst. Mining and Metallurgical Engrs., February, 1929.

[3] R. R. Moore, "Effect of Corrosion upon the Fatigue Resistance of Thin Duralumin," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 128 (1927).

[4] F. N. Speller, I. B. McCorkle and P. F. Mumma, "Influence of Corrosion Accelerators and Inhibitors on Fatigue of Ferrous Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 159 (1928).

[5] The term "duralumin" is used throughout as referring to the class of heat-treatable aluminum alloys in which the essential alloying elements are copper, magnesium, silicon and manganese, and not to the product of any particular manufacturer.

The work discussed here is a part of this investigation, which is still in progress. A detailed résumé of the previous work is hardly necessary here. The early phases of the work were concerned with laboratory corrosion tests which later were supplemented and confirmed by weather-exposure tests, tension specimens being used throughout as the corrosion specimens. The change in the tensile properties in the material after corrosion afforded the most useful means for evaluating the effect of the corrosive attack. It has been definitely shown that the embrittlement of sheet duralumin referred to above is essentially a corrosion problem, the corrosive attack generally being intercrystallin in character. The most important factors affecting the susceptibility of the alloy to this form of deterioration relate to the treatment rather than to the composition of the material. Heat treatment, consisting of quenching followed by aging, is essential in obtaining the high-strength properties of the duralumin type of alloy. Material which has been quenched in cold water is more resistant to the attack than if hot water is used as the quenching medium. Either one may be used with equal success in obtaining the desired high strength and ductility, however. Likewise, material aged at room temperature, after quenching, is more resistant than material aged at elevated temperature. The efficacy and value of coatings and other measures to protect the material against the attack depend upon circumstances. For marine atmospheric conditions, aluminum, according to present knowledge is, by far, the most dependable coating. In considering the tests reported upon here these facts should be borne in mind, since the test materials used were selected on the basis of the results of the previous series of tests.

It should be emphasized at the outset that the tests reported upon are primarily corrosion tests. Although in part of the work the material was subjected to cyclic flexural stresses, the primary aim has not been to determine the life of the material under such conditions but rather to show how and to what extent the corrosion behavior, and hence the usefulness, is dependent upon the imposed conditions. The results of endurance or fatigue testing of the same material of course form a valuable background against which the present results may be viewed.

MATERIAL AND METHOD OF TEST

Material:

Nearly all of the tests were carried out upon a duralumin type of alloy commercially available under the name "17ST".[1] The average composition of the material used was:

Copper, per cent	4.1
Iron, per cent	0.34
Silicon, per cent	0.32
Manganese, per cent	0.51
Magnesium, per cent	0.61
Aluminum, per cent	remainder

The average tensile strength of the alloy, in the form of No. 14 gage (0.064 in.) sheet, was as follows:

Tensile Strength, lb. per sq. in	59,000 to 61,000
Yield Point (stress for 0.006 in. per inch extension under load), lb. per sq. in	40,000 to 42,000
Elongation in 2 in., per cent	20 to 23

Some No. 16 gage (0.052 in.) sheet was also used. Much of the material was aluminum clad sheet,[2] a commercial material in which the coating forms an integral part of the finished sheet which is formed by rolling, into sheet form, a composite slab, consisting of duralumin with aluminum on the two opposite faces. The thickness of the two aluminum surface layers for the No. 14 gage sheet used was approximately 5 per cent of the total thickness of the sheet. The tensile strength of the composite sheet is, of course, slightly less than that of a plain sheet of duralumin of the same thickness. The following properties are typical:

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[1] The materials were generously supplied by the Aluminum Co. of America.

[2] E. H. Dix, "Alclad, a New Corrosion-Resistant Product," *Technical Note 259*, National Advisory Committee for Aeronautics (1927).

Tensile Strength, lb. per sq. in	55 000
Elongation in 2 in., per cent	19 to 22

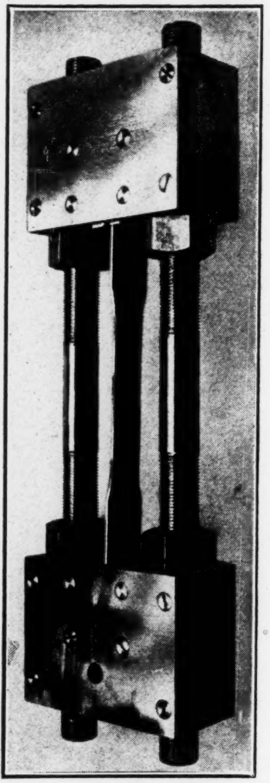

FIG. 1.—Device for Maintaining a Specimen Under Tension During Corrosion, With Specimen in Position. The Assembled Device After Covering the Surface With Paraffin, Except that of Central Part of the Specimen, is Used as the Corrosion Specimen.

Fatigue Characteristics of the Two Materials.—The endurance limit of heat-treated duralumin sheet, as determined by repeated flexure, is, in all probability, close to 15,000 lb. per sq. in. The corresponding value for aluminum-clad duralumin sheet is somewhat lower.[1] The corrosion-fatigue limit of duralumin as given by

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[1] The Bureau of Standards investigation, still in progress, indicates that this value is slightly above 10,000 lb. per sq. in.

McAdam[1] is, in fresh water, 7000 to 9000 lb. per sq. in., and in salt water 6000 to 8000 lb. per sq. in.

A few comparison tests were also carried out upon a high-strength aluminum alloy sheet "51ST," an alloy which is almost entirely free from copper and owes its characteristic properties to the presence of magnesium and silicon.

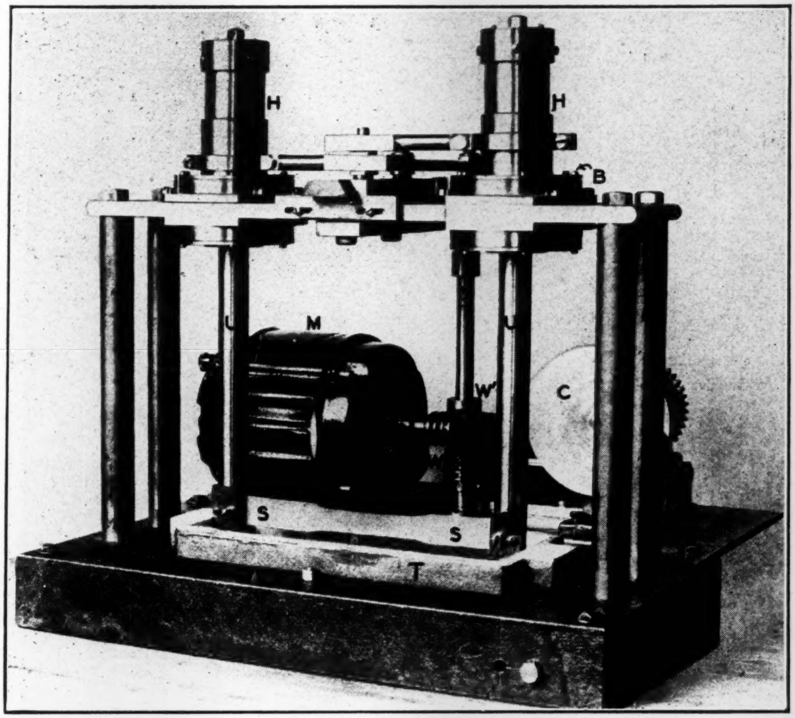

FIG. 2.—Side View of the Apparatus Used for Corroding a Specimen While it is Repeatedly Stressed by Flexure.

M is the motor.
W and *W'* are the worm gears.
C is the cam.
T is the tank containing solution which is raised at 15-minute intervals.
S is the specimen.
U is the upright with the Cardan suspension at the top inside the housing *H*.
B is the ball bearing.

Methods:

Two methods were used: (*a*) the specimen was maintained under tension while being corroded; and (*b*) the specimen was stressed repeatedly by flexure while being corroded. Ten-inch tension-test bars were used and the tensile properties after corrosion periods of various durations were determined. The corrosion method used in these tests, as well as in the earlier ones in the investigation, was the wet-and-dry or repeated immersion test, such a test being considered more comparable to conditions which prevail in service than a continual wetting of the surface would be.

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[1] D. J. McAdam, Jr., "Corrosion Fatigue of Non-Ferrous Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 102 (1927).

(*a*) *Static Tension.*—The device used for corroding the specimens while held in tension is shown in Fig. 1. By means of tapped holes (not shown in the photograph) in the ends of the two end pieces, into which suitable threaded fixtures could be screwed, it was an easy matter to hold the device in the testing machine while applying a load of any desired amount. The nuts on the two threaded bars, which fit rather loosely through the two end pieces, were used to maintain the applied load. All surfaces, except the central portion of the specimen, were then coated with paraffin and the entire device used as the corrosion specimen for the "wet-and-dry" corrosion test. The apparatus in its early form has already been described,[1] and a

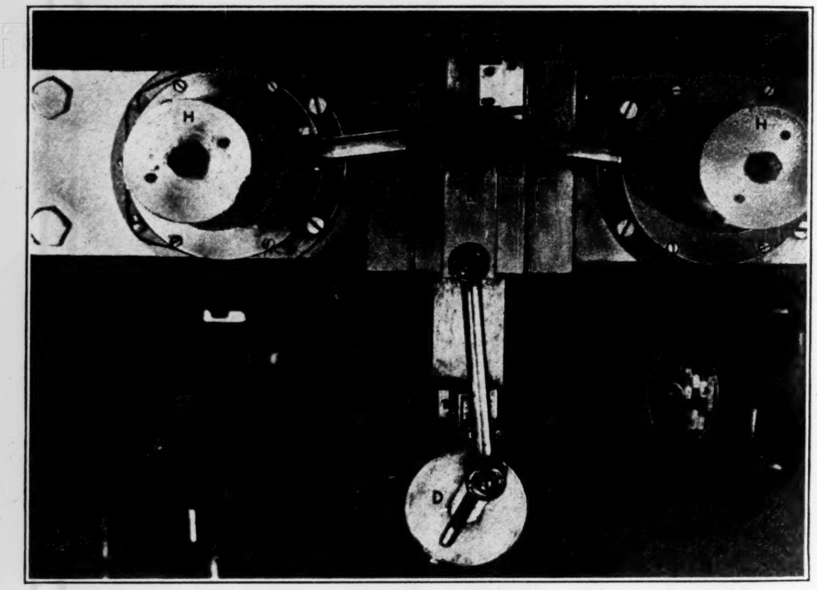

FIG. 3.—Top View of the Apparatus Shown in Fig. 2.

H is the housing within which are the Cardan suspensions for the uprights.
R is the reciprocating slide.
D is the rotatory drive actuated by the worm drive shown in Fig. 2.

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[1] H. S. Rawdon, A. I. Krynitsky and W. H. Finkeldey, "Types of Apparatus Used in Testing the Corrodibility of Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, p. 717 (1924).

detailed description need not be given here. In brief, the method consisted in the immersion of the specimen at 15-minute intervals in a normal solution of sodium chloride (approximately 5.5 per cent by weight) to which had been added an amount of commercial hydrogen peroxide solution equal to one-tenth of the volume of the whole. The specimen was only momentarily immersed in the solution; for the greater part of the 15-minute period it was suspended horizontally in the air. After being corroded for the desired period, the specimen was removed from the holder and tested in tension.

(*b*) *Repeated Flexure.*—The apparatus, by means of which the corrosion of the specimens while they were being repeatedly stressed by flexure was carried out, is shown in Figs. 2 and 3.[1] The specimen was securely bolted at the ends to two uprights which could be rotated about their longitudinal axes and thus bend the specimen. The suspension for the uprights used was that usually referred to as a "Cardan support" which consists essentially of two X-shaped spring members, one above the other, placed 90 deg. to each other. This method of suspension which is sufficiently strong to transmit the necessary torque, permits freedom of motion transversely in all directions of the lower part of the upright. By this means, no significant stresses other than those resulting from bending were set up in the specimen. The specimen was flexed at approximately 75 complete bends per minute.[2]

At intervals of 15 minutes the enameled cast-iron tank (*T*, Fig. 2) containing the corrodent was automatically raised sufficiently to immerse the specimen and held for approximately a minute. For purposes of comparison, a second specimen was suspended freely by the side of the stressed one and corroded under the same conditions. The tensile properties of the corroded bars were then determined.

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[1] The machine was constructed by J. Ludewig, Mechanician, the essential features, especially the suspension of the uprights, being suggested by L. B. Tuckerman, Engineer Physicist, Bureau of Standards.

[2] The calibration of the machine by a method suggested by L. B. Tuckerman utilized the principle of auto-collimation of a beam of light. Directly in front of two totally reflecting glass prisms attached to the flat side of the specimen (2 in. apart) and facing each other, were placed two similar convex lenses. By such an arrangement a beam from a light source situated in front of one lens at a distance equal to the focal length of the lens will be reflected by means of the two prisms in succession and emerge from the second lens parallel to its initial path, when the specimen is at rest and unstrained. A suitable horizontal scale can be used to locate the position of the returning beam. As the specimen is flexed the spot of light travels across the scale and back. The data furnished by this set-up, together with Young's modulus of elasticity for the material, are sufficient for the calculation of the maximum fiber stress of the flexing strip. The equation used for this was:

$$\text{maximum fiber stress, lb. per sq. in.} = \frac{tsE}{4fl}$$

in which t = thickness of the specimen, f = focal length of the lens, l = distance between prisms (2 in.), s = scale reading (total swing of spot across the scale), and E = Young's modulus (10,000,000 lb. per sq. in., assumed for duralumin).

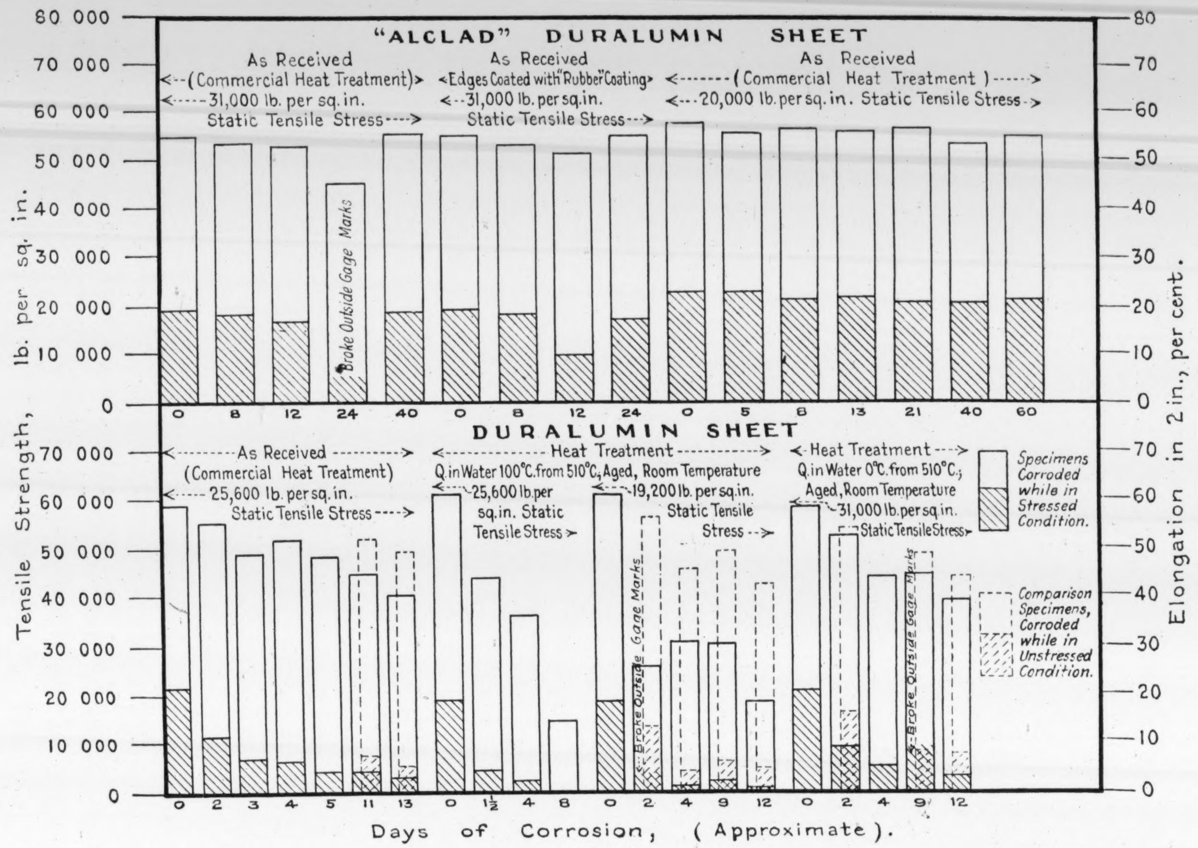
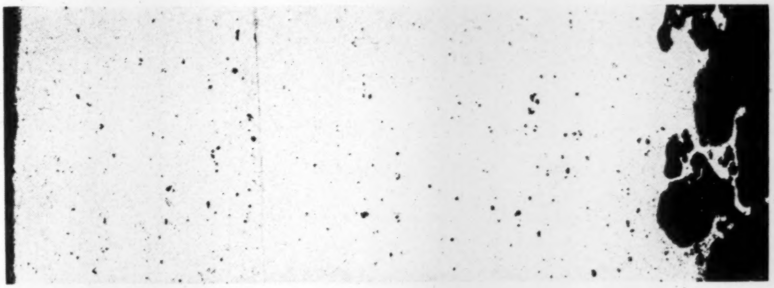


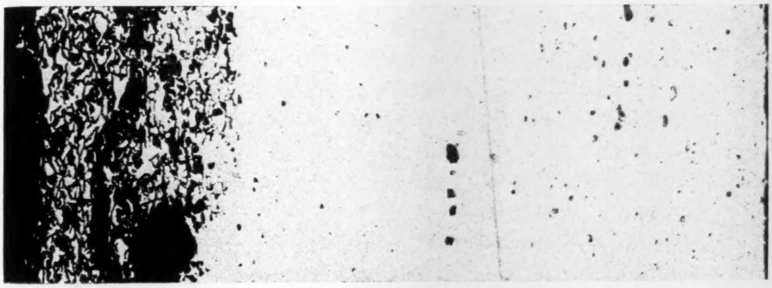
FIG. 4.—Effect of Static Tension, Acting Simultaneously with Corrosion, on the Tensile Properties of Sheet Duralumin, Both Plain and Aluminum-Clad.

The corrosion was carried out by the wet-and-dry method by immersion at 15-minute intervals in a sodium-chloride, hydrogen-peroxide solution. **The full height of the rectangle in each case indicates the tensile strength; the height of the cross-hatched portion indicates the elongation of the stress-corroded specimen. The properties of corresponding comparison specimens are shown by the figure in dotted lines.**



(*a*) Cold-water-quenched duralumin sheet after 4 days' attack, 30,800 lb. per sq. in. tensile stress.

(*b*) Same as (*a*) after 12 days.



(*c*) Hot-water-quenched duralumin sheet after 4 days' attack; 19,800 lb. per sq. in tensile strength.

(*d*) Same as (*c*) after 12 days.

FIG. 5.—Microstructure of No. 16 Gage Sheet Duralumin Corroded While Under Tension. Longitudinal Sections Perpendicular to the Flat Side of the Specimen, Unetched, in all Cases (× 75).

The photomicrographs reproduced here were originally taken at magnifications of 90 but were reduced to 75 in reproduction.

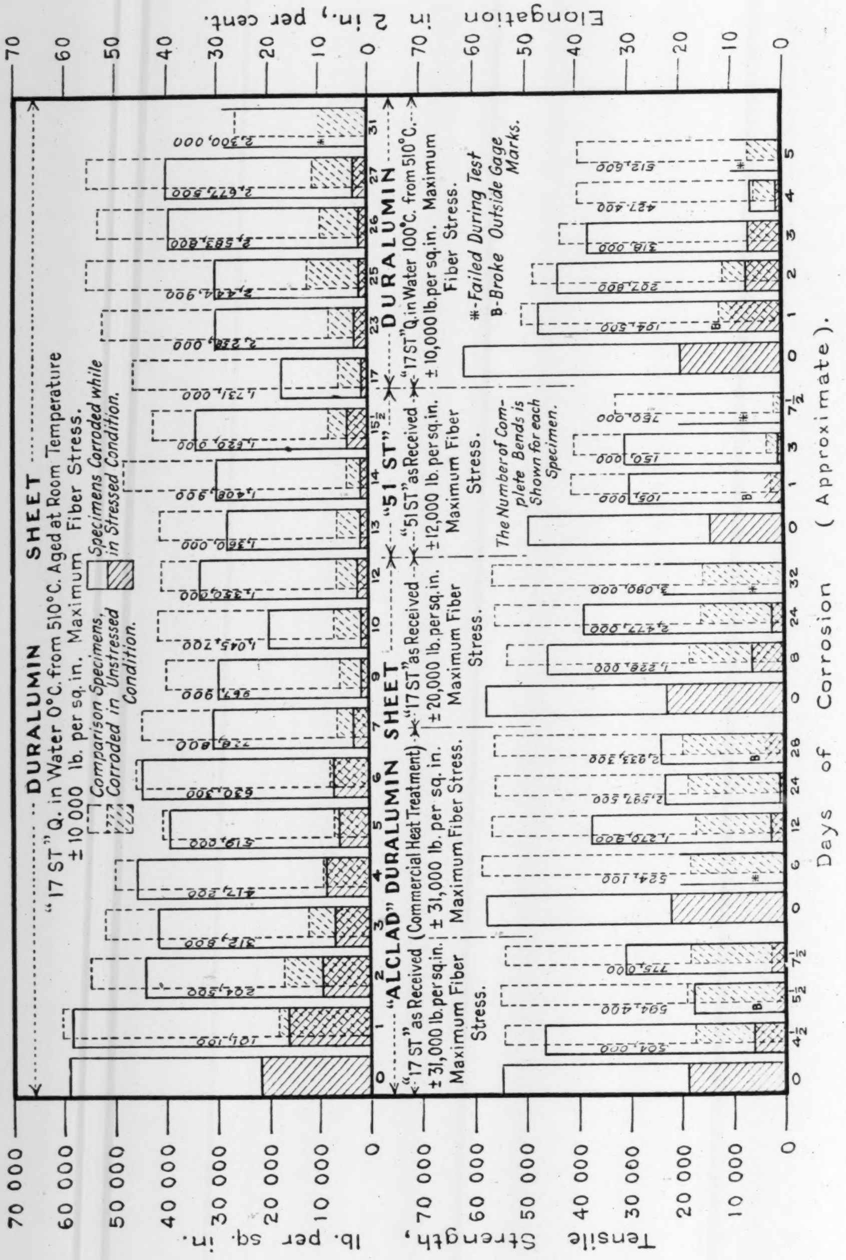
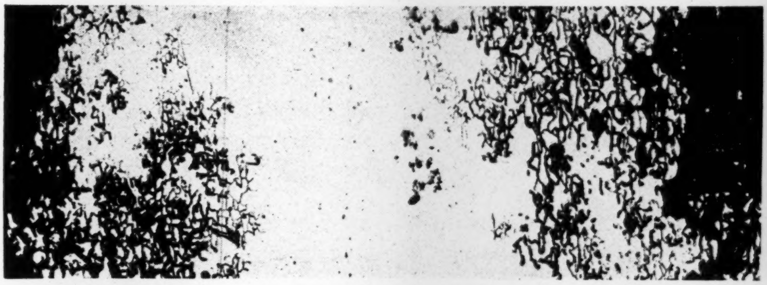
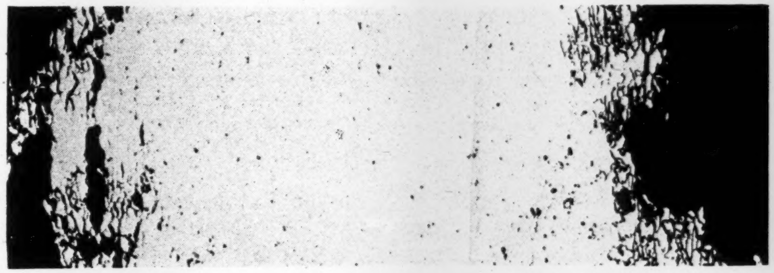


FIG. 6.—Effect of Repeated Flexural Stress, Simultaneous with Corrosion, on the Tensile Properties of Sheet Duralumin, Both Plain and Aluminum-Clad. Results are also Given for Another High-Strength Aluminum Alloy, "51ST."

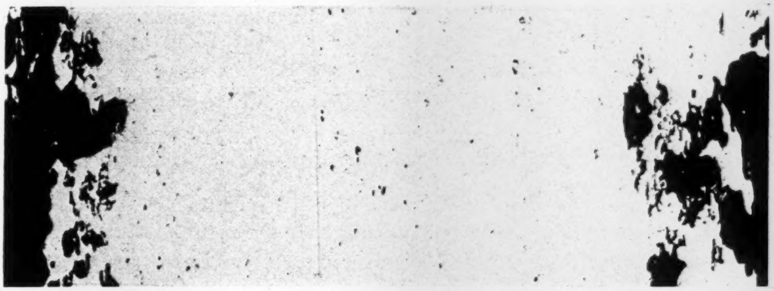
The corrosion was carried out by the wet-and-dry method by immersion at 15-minute intervals in a sodium chloride - hydrogen peroxide solution. The full height of the rectangle in each case indicates the tensile strength; the height of the cross-hatched portion indicates the elongation of the stress-corroded specimen. The properties of corresponding comparison specimens are shown by the figure in dotted lines.



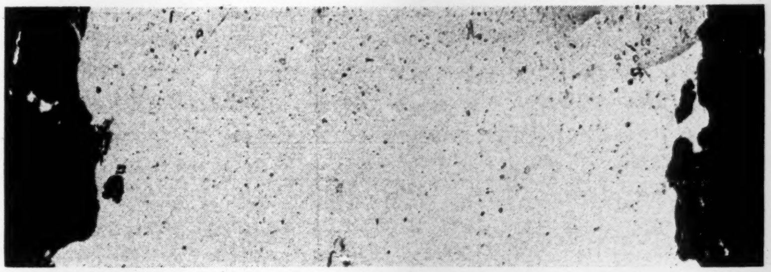
(*a*) Hot-water-quenched sheet duralumin after 4 days' attack.



(*b*) Similar material, unstressed, after 5 days' attack.



(*c*) Cold-water-quenched duralumin sheet after 3 days' attack.



(*d*) Similar material after 9 days' attack.

FIG. 7.—Microstructure of No. 16 Gage Sheet Duralumin Corroded While Being Repeatedly Stressed by Flexure, 10,000 lb. per sq. in. Maximum Fiber Stress. Longitudinal Sections Perpendicular to the Flat Side of the Specimen, Unetched, in All Cases (× 75).

The photomicrographs reproduced here were originally taken at magnifications of 90 but were reduced to 75 in reproduction.

With a few of the specimens, however, the corrosive attack was severe enough to result in failure of the specimen in the machine.

TEST RESULTS

According to the results of previous tests, aluminum-coated duralumin appeared to be the most dependable material of the high-strength aluminum alloy class. This material was therefore given preference over a number of others which might well have been studied. Likewise, since the heat treatment of duralumin sheet appears to be an important factor in determining its susceptibility to corrosive attack, this was borne in mind in selecting the materials for tests.

Corrosion Accompanied by Static Tension:

The results of the static tests are shown in Fig. 4. According to these results, duralumin sheet, coated with aluminum, maintains its initial properties unimpaired for corrosion periods as long as 60 days with an applied tensile stress as high as 20,000 lb. per sq. in. Even with a stress considerably higher than this (31,000 lb. per sq. in.), the properties were not seriously impaired after 40 days' attack.

On the other hand, a few days' attack of the unprotected alloy sheet under corresponding conditions was sufficient to cause a pronounced lowering of the tensile properties. In these tests, the material which had been heat-treated by being quenched in cold water was superior to the sheet material which was heat treated by being quenched in hot water, though far inferior to similar material having the aluminum coating. These results are in excellent agreement with the results of previous laboratory and exposure tests.

In Fig. 5 is shown the microstructure of sheet duralumin after being corroded while in the stressed condition. It will be noted that the attack is, on the whole, typically intercrystalline in its nature. The effect of the applied stress was to accelerate the corrosive attack but not to change its character in any essential respects as shown by the previous tests on unstressed specimens.

Corrosion Accompanied by Repeated Flexure:

The corrosive attack which occurred under repeated flexure was the most severe which has been encountered in any of the tests carried out. A corrosion test carried out in this manner is, indeed, a "searching" one and the dependability in service of a material showing superior corrosion-resisting properties under such conditions cannot well be questioned. The results are summarized in Fig. 6. The examination of the microstructure (Fig. 7) illustrates the fact that the characteristic features of the corrosive attack of the plain duralumin were not changed by the application of cyclic stress during the attack.

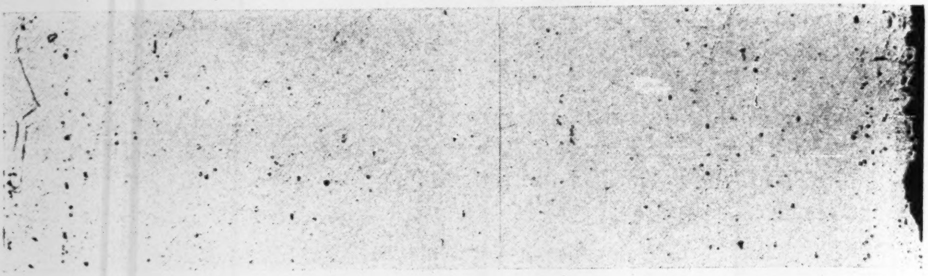
DISCUSSION

The results obtained in the foregoing tests in which the specimens were corroded while stressed in tension unmistakably show the relative merits of the two classes of material, plain duralumin sheet and the aluminum-clad duralumin sheet. The latter, when stressed one-half the yield point (20,000 lb. per sq. in. stress for the coated sheet against 40,000 lb. per sq. in. for the yield point of duralumin sheet) showed no pronounced or significant change in its tensile properties after as much as 60 days' exposure to severely corrosive conditions. The same material stressed as high as 31,000 lb. per sq. in., while being corroded, showed tensile properties after 40 days' corrosion which were only slightly below the initial properties. According to the test results, protection of the exposed or cut edges of the aluminum-coated sheet is unnecessary.

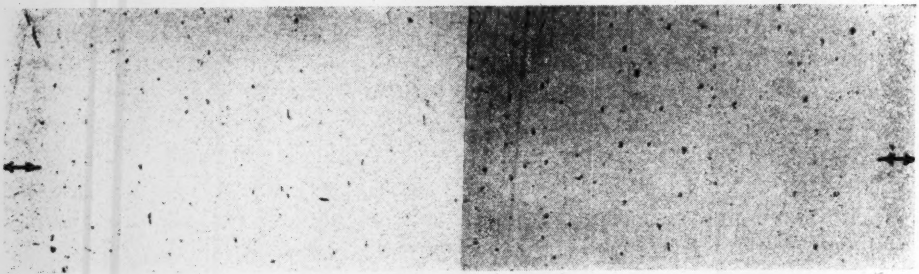
On the other hand, the plain duralumin sheet, suitably heat-treated, when corroded under the same conditions showed a pronounced drop in the tensile properties after 2 days' attack and after 12 days was very severely attacked. The same material when heat-treated by hot-water quenching was still more severely attacked even under a somewhat lower stress. It should be noted, however, that even in the unstressed condition, the corrosive attack of plain duralumin sheet was severe, and the additional effect produced by stressing the material in tension was considerably less than the effect of corrosion alone.

On the basis of the present test results, it may be questioned whether the accelerating effect of static tensile stress on susceptibility to corrosion need be considered very seriously if the conditions are mild enough to warrant the consideration of the use of plain (uncoated) duralumin sheet, as, for example, in inland service far removed from marine conditions. Certainly if the material can be given a coating which will not crack or flake under the stress acting, as illustrated by the aluminum-coated sheet, the effect of a static tensile stress, even of relatively high magnitude, may be considered entirely negligible so far as corrosion acceleration is concerned.

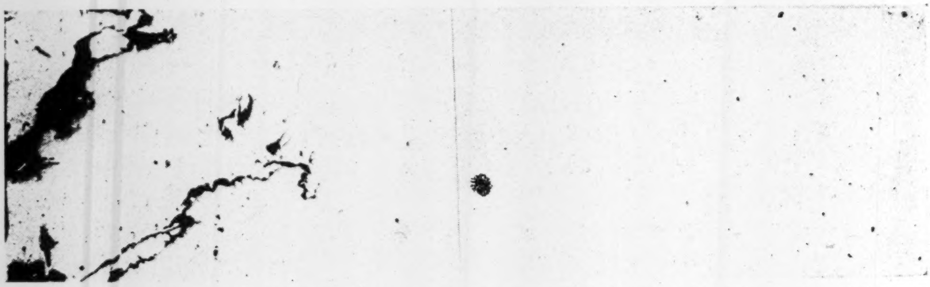
It is important to note that the effect of tensile stress, acting simultaneously with corrosion, was not such as to change the characteristic nature and was especially pronounced in the hot-water-



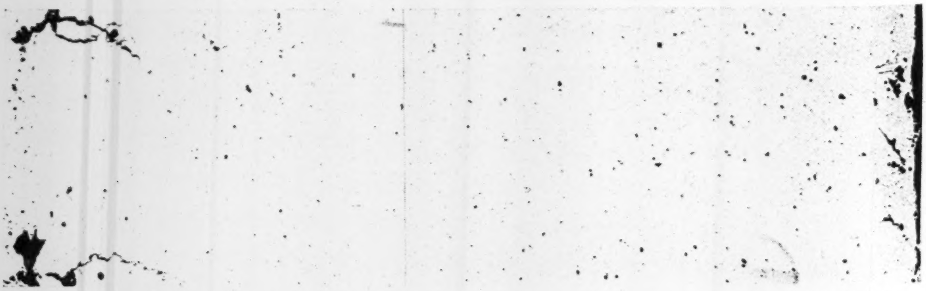
(*a*) Maximum stress ±20,000 lb. per sq. in., for 24 days, total number of complete bends 2,477,000. Note the cracks which have formed in the aluminum layer.



(*b*) Comparison specimen, unstressed, corroded under same conditions as (*a*). The thickness of the aluminum coating is indicated.



(*c*) Same stress as (*a*), 32 days, total number of complete bends 3,080,000. The specimen failed during the "run." Note the 45-deg. trend of the cracks.



(*d*) Maximum stress ±31,250 lb. per sq. in., for 6 days. The specimen failed during the "run." Total number of complete bends 524,130.

Fig. 8.—Microstructure of Aluminum-Clad Duralumin Sheet After Corrosion Accompanied by Repeated Stressing by Flexure. Longitudinal Sections Perpendicular to the Flat Side of the Specimen, Unetched, in All Cases (× 75).

The photomicrographs reproduced here were originally taken at magnifications of 90 but were reduced to 75 in reproduction.

quenched material. In the cold-water-quenched sheet the intercrystalline attack was in large measure obscured by pitting in the later stages of the attack.

The relatively long life shown by the cold-water-quenched duralumin sheet may be somewhat misleading unless due and careful consideration is given to the tensile properties, both tensile strength and ductility, of the material during the progress of the test. During the early stages, the corrosive attack showed its greatest effect in the lowered ductility; this dropped to a very low value and remained almost unchanged during the latter half of the test. The tensile strength, however, continued to drop, with but few exceptions, throughout the latter part of the test. The effect of the imposed stress in accelerating the corrosive attack is best shown by a comparison of the tensile strengths of the two sets of specimens, stressed and unstressed, rather than by the ductility. The reduction in tensile strength is indicative of the reduced cross-section of the material, that is the depth to which corrosion has penetrated, whereas the ductility is apparently influenced more by the manner of the corrosive attack. During the early stages of the corrosion of the cold-water-quenched sheet, the intercrystalline attack predominated and a penetration of the metal by this method, even if only to a slight depth, was sufficient to reduce the ductility to a relatively low value. As the corrosion was continued, however, the attack assumed the form of pitting and as the cross-section was progressively decreased by the deepening of the pits, the tensile strength decreased in like manner. Although, as indicated by the number of bends which the specimens withstood for the severe conditions used($\pm 10,000$ lb. per sq. in. + severe corrosion), the life of the material might be stated to be in the neighborhood of 3,000,000 complete bends, it is evident from a consideration of the progressive decrease in the tensile properties (Fig. 4) that life for which the material could be recommended for these conditions would be decidedly less than this value.

Since the hot-water-quenched duralumin is more susceptible to the intercrystalline type of attack, this continued throughout as the predominating form of corrosion. As shown by the micrographs of the corroded specimens (Fig. 7) a relatively deep penetration of the metal together with failure of the specimen resulted relatively early.

The aluminum-clad duralumin sheet in all cases was stressed to a degree much higher than the stress corresponding to the endurance limit of this material. In spite of this fact, however, a life of 24 to 32 days was shown by this material under the very severely corrosive conditions which were imposed upon it. This life was, of course, directly determined by the behavior of the coating. Not until failure of the coating occurred was there any noticeable attack of the underlying duralumin. The examination of the microstructure of the tested specimens suggests (Fig. 8) that the fatigue stresses played a much more important part in the failure than did corrosion. The cracks which formed in the aluminum surface layer and gradually extended into the underlying duralumin as the flexing of the bar was continued were very similar in appearance to the corresponding cracks which form in the same material when it is repeatedly stressed in the absence of a corrodent, as in the air. In many cases the cracks showed a definite tendency to form at an angle of 45 deg. with the surface, a feature in which corrosion would play little or no part. The rate at which the cracks progressed may, of course, have been affected by the corrodent. It is of interest to note that the characteristic intercrystalline type of corrosive attack was not at all pronounced in the specimens.

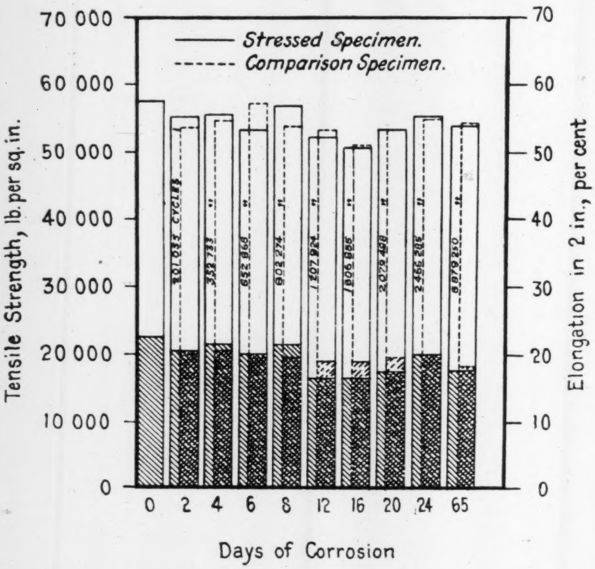


FIG. 9.—Effect of Repeated Flexural Stress, Simultaneous with Corrosion, on the Tensile Properties of Aluminum-Coated Duralumin. Compare with results of plain duralumin, Fig. 6.

It is apparent from the above tests that aluminum-coated duralumin is far superior to plain duralumin in its resistance to the combined effect of stress and corrosion even though the applied stress be relatively very high. This fact is much more strikingly shown, however, by the results summarized in Fig. 9 in which the applied flexural stress was identical with that used for the plain duralumin (10,000 lb. per sq. in.) and thus more nearly comparable to the normal service stresses. It will be noted that the aluminum-coated duralumin sheet was not appreciably affected in its tensile properties by corrosion accompanied by stress for a period as long as 65 days.

It is evident that when a corrosive attack is accompanied by cyclic stress, the relative importance of the effect produced by each will, for a given stress value, be largely dependent upon the frequency of the stress cycle used. McAdam[1] has discussed some aspects of this phase of the general stress-corrosion problem. Since the present tests were primarily corrosion tests, the corrosion factor was made the predominating one by using a relatively low frequency in the application of the flexural stress. Such a combination of the corrosion and the stress factors as this one is also believed to be more truly representative of service conditions, in which the limit of the usefulness of a material is determined by corrosive influences rather than by the stress acting, than similar tests in which the stress factor predominated, such as would be the case if the tests more nearly approximated fatigue tests.

Acknowledgments.—The tests which form the basis of this report were carried out by R. W. Buzzard, Junior Scientist, and W. H. Mutchler, Scientific Aid, to whom the author wishes to acknowledge his indebtedness.

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[1] D. J. McAdam, Jr., "Corrosion of Metals as Affected by Stress, Time and Number of Cycles," *Technical Publication No. 175*, class E, Inst. of Metals, No. 62, Am. Inst. Mining and Metallurgical Engrs. Issued with *Mining and Metalluogy*, February, 1929.

DISCUSSION

Mr. T. W. Downes[1] (*presented in written form*).—The work of Mr. Rawdon at the Bureau of Standards on the corrosion of light alloys, results of part of which form the subject matter of the paper under discussion, has been followed with intense interest by the Bureau of Aëronautics of the Navy Department inasmuch as the corrosion in service of duralumin and aluminum-coated duralumin naval aircraft parts is of vital importance.

Although no stress-corrosion tests of light alloys have been made at the Naval Aircraft Factory, the relation of the rapidity with which duralumin is quenched from the heat treatment temperature (500 to 510° C.) to the rate of corrosion as determined by visual examination and the reduction of the tensile properties has been studied. The results confirm the statement of Mr. Rawdon that duralumin which has been quenched in cold water is more resistant to corrosive attack than that quenched in hot water. Further than this the results show that when quenched in coolants held at temperatures below and above the range covered by cold and hot water, duralumin is more and less resistant, respectively, to corrosion by salt water spray than when quenched within this range.

The work done at the Naval Aircraft Factory to which reference is made above was carried out on two thicknesses of duralumin sheet, namely, $\frac{1}{16}$ and $\frac{1}{8}$ in., the object being to investigate the effects upon the physical properties, rate of aging and resistance to corrosion of cooling from the quenching temperature by the following methods:

1. Iced brine (10° F.).
2. Ice water (34° F.).
3. Water at 63° F.
4. Air spray (72° F.) (water atomized by compressed air).
5. Direct water spray (74° F.).
6. Boiling water (212° F.).
7. Steam at 15 lb. gage pressure (250° F.).
8. Slow cooling in the furnace.

Space does not permit giving here a detailed account of the test procedure and of all results, but a brief statement regarding the corrosion results is believed warranted. As judged by the visible corrosion produced by continuous exposure for 60 days in the spray of a 20-percent water solution of sodium chloride the specimens indicated that the above order of the various coolants obtained for the $\frac{1}{8}$-in. thick material for increasing corrosive attack, that is, specimens quenched in the iced brine were corroded least and those quenched in steam were the most severely attacked. Manifestation of corrosion in this time ranged from only a few, small, shallow pits for the iced-brine-quenched specimens to numerous fine and relatively large pits for the steam-quenched specimens. Approximately, the same order for the various coolants was indicated by the $\frac{1}{16}$-in. thick specimens. The specimens which were slowly cooled in the furnace were very much more severely corroded than the steam-quenched specimens, but these, of course, represented the annealed condition of the metal. It is interesting to note that the tensile properties of the various specimens, when tested after the corrosion exposures, confirmed approximately, as regards both strength and elongation, the estimated degree of the corrosive attack. Unfortunately examination for extent of intercrystalline corrosion was not made.

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[1] Metallurgist, U. S. Naval Aircraft Factory, Philadelphia Navy Yard, Philadelphia, Pa.

Although these tests were made on specimens which were heat-treated and then corroded in the unstressed condition prior to testing it is believed reasonable to assume that the more severe the quench the greater will be the time required for failure under a constant static load or repeated flexure when accompanied by corrosion or, conversely, the greater will be the load required to produce failure in a given time or by a given number of repeated stresses. This is thought to be true because of the fact, as Mr. Rawdon points out, that examination showed that the application of neither static nor cyclic stress changed the character of the corrosive attack on the duralumin during the period of attack.

Mr. Rawdon has shown that cold-water-quenched duralumin is less susceptible to the intercrystalline type of attack than the hot-water-quenched material. On the assumption that the more drastic the quench the more resistant is the material to this form of corrosion it would be interesting to investigate the behavior of duralumin quenched in iced brine or other lower temperature coolants when subjected to corrosion under conditions of static, flexural, or other types of cyclic stress.

MR. H. F. MOORE.[1]—It seems to me that Mr. Rawdon's paper has a very direct bearing on the paper being presented by Messrs. Johnson and Oberg.[2] In the Johnson-Oberg paper we have a rather pessimistic evaluation of the endurance limit for duralumin, while in his discussion Mr. McAdam is more optimistic. Mr. Rawdon shows how sensitive duralumin is to some corrosive agencies, and it occurs to me that the point raised by Mr. McAdam and Mr. Templin in discussion of the Johnson-Oberg paper is very well worth consideration, namely, that over long periods of time the effect of *air* corrosion on the fatigue strength of duralumin may be considerable. A recent paper by Parker Haigh of the Royal British Naval Academy[1] states that he found that for certain metals, especially for soft metals which show discoloration on fracture, the endurance limit may be appreciably raised by protection against air corrosion.

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[1] Professor of Engineering Materials, University of Illinois, Urbana, Ill.

[2] J. B. Johnson and T. T. Oberg, "Fatigue Resistance of Some Aluminum Alloys," see p. 339; discussion, p. 344.

In very long-time tests the possibility of occasional wetting of the specimens by rain through open windows or through a leaky roof should be considered; on the latter point the speaker has had sad experience. In any event Mr. Rawdon has shown us that corrosion is a factor of no small importance in determining the practical strength of duralumin.

MR. E. H. DIX, JR.[2]—Mr. Rawdon's paper is so full of interesting material that can be applied practically that I find there are a number of things I should like to say.

Mr. Rawdon has brought out quite clearly that these results are laboratory tests, run under very extreme conditions. In our own work we have found that the sodium chloride - hydrogen peroxide solution which Mr. Rawdon used is so effective in promoting intergranular attack on the strong aluminum alloys that the loss in tensile strength and elongation caused by a 48-hour intermittent immersion is equivalent to perhaps the loss obtained in three months in the continuous salt spray. We have no way at present of saying how long that means in ordinary atmospheres.

I think we should stress also that this work is on a relatively thin sheet which is not protected in any way. I think that all users of the strong aluminum alloys realize now that thin sheets of these alloys should be protected, especially when using a salt atmosphere, and undoubtedly the paint and varnish coverings have been worked out to the point where very reliable protection is obtained in that way.

In connection with the results shown on the "Alclad" material, Mr. Rawdon has already indicated that the stresses of 31,000 lb. per sq. in. shown in the first table are so high that the failure may have resulted from fatigue as well as corrosion. Personally, I think it is almost altogether a matter of fatigue, and I should like to point out that in calculating this stress of 31,000 lb. per sq. in., which, of course, is supposed to represent the stress in the outer fiber, the formula which is used for a homogeneous material has been used also in the case of the "Alclad" material. The outer surface, the $5\frac{1}{2}$ per cent on each side is, of course, soft aluminum, which has a relatively low strength.

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[1] Parker Haigh, "Chemical Action in Relation to Fatigue in Metals," a paper read before the British Institution of Chemical Engineers on March 20, 1929.

[2] Metallurgist, Research Bureau, Aluminum Co. of America, New Kensington, Pa.

There is some question as to how much error this introduces in the computation, but it has been calculated to be as high as 25 per cent. Now if you add 25 per cent to the 31,000, you will see that the stress is well above the yield point of the material.

Another point that is interesting to me are the micrographs of the fatigue specimens of the "Alclad" sheet, where there is shown a crack starting in the pure aluminum surface and later extending into the strong alloy core. Mr. Rawdon commented that there was no indication of inter-granular corrosion. I think it is proved rather conclusively that the presence of the pure aluminum exerts an electrolytic protection over the alloy, that is, the pure aluminum tends to go into solution and in so doing protects the alloy underneath.

We have made rather conclusive tests which show this to be true and, therefore, it is quite reasonable that there should be no intergranular corrosion, even though the surface is broken through and cracks start into the duralumin.

Just one other point, in connection with the discussion of the paper by McAdam,[1] it was suggested that the scatter points in fatigue tests on normal duralumin might be caused by corrosion. In some work which the Bureau of Standards has been doing, which is yet unpublished, it was quite evident that there was very much less scatter of points in the curve for "Alclad" material than for the normal material. Of course, my first thought was that it was the corrosion that was causing the scatter in the duralumin. However, we subjected some fatigue specimens, that is, the finished machined specimens to salt spray for a period as long as six months, and we were quite surprised to find how little this treatment reduced the fatigue strength. I believe Mr. H. F. Moore or Mr. Gillette of the Bureau of Standards, or perhaps both, made the suggestion that surface imperfections will have a smaller effect in reducing the fatigue strength where there is a soft aluminum surface. That is, in the case of "Alclad" material the strength resides in the core, and a small scratch or abrasion on the pure aluminum surface would have a relatively small effect on the fatigue properties. It may be that it is a question of preparing the specimen, preventing a scratch or small abrasion, in order to prevent the scatter of points in the fatigue curves of the normal materials.

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[1] D. J. McAdam, Jr., "Corrosion of Metals Under Cyclic Stress," see p. 250; discussion, p. 304.

MR. D. J. MCADAM, JR.[1]—Mr. Rawdon's paper is a very valuable one, especially the information presented about "Alclad." In the last paragraph of his paper, however, he refers to a paper of mine[2] and uses it to support a conclusion that I think is not correct. In regard to other parts of the paper also further comment is desirable.

In estimating the results of corrosion tests he uses tension tests whereas at the Naval Engineering Experiment Station we have made fatigue tests on the previously corroded specimens.[2,3,4,5] Either method will give information of value, but in using the tension test method care is necessary to avoid entering what I have called the "second period of corrosion-fatigue."

In some of Mr. Rawdon's experiments, specimens were corroded under alternating stress. As I have shown in several recent papers,[2,3,4,5] the effect of corrosion under such conditions is to lower the fatigue limit of the corroded specimen more rapidly than if it were subjected to stressless corrosion. This lowering of the fatigue limit is due to stress concentration at the bottoms of corrosion pits. The progress of pitting in the first stage of corrosion-fatigue, however, is due to corrosion, not to ordinary fatigue, since the actual stress at bottoms of corrosion pits is below the endurance limit.

To represent this relationship a curve may be drawn with corrosion time as abscissas and resultant fatigue limits as ordinates. The origin of this curve for duralumin is at an ordinate representing the endurance limit of the metal, about 15,000 lb. per sq. in. The curve descends with gradually decreasing slope until the ordinate is 10,000 lb. per sq. in., numerically the same as the alternating stress applied during corrosion. At this point the actual stress at the bottoms of the corrosion pits reaches the endurance limit of the metal. With further corrosion under cyclic stress the progress of the pits is due to fatigue as well as to corrosion. Hence from this point the curve descends with increasing slope.

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[1] Metallurgist, U. S. Naval Engineering Experiment Station, Annapolis, Md.

[2] D. J. McAdam, Jr., "Corrosion of Metals as Affected by Stress, Time and Number of Cycles," *Technical Publication No. 175*, Class E, Inst. of Metals, No. 62, Am. Inst. Mining and Metallurgical Engrs. Issued with *Mining and Metallurgy*, February, 1929.

[3] D. J. McAdam, Jr., "Corrosion of Metals as Affected by Time and by Cyclic Stress," *Technical Publication No. 58*, Class E, Inst. of Metals, No. 21, Am. Inst. Mining and Metallurgical Engrs. Issued with *Mining and Metallurgy*, February, 1928.

[4] D. J. McAdam, Jr., "Some Factors Involved in Corrosion and Corrosion-Fatigue of Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 117 (1928).

[5] D. J. McAdam, Jr., "Corrosion of Metals Under Cyclic Stress," see p. 250.

In investigating the effect of cyclic stress on corrosion, obviously, the proper field of investigation is limited to the first period of corrosion-fatigue. The second period of corrosion-fatigue must be avoided, as in that period ordinary fatigue is a complicating factor.

If the resultant fatigue limit be used as a criterion of the damage due to corrosion, it is easy to determine whether or not the "second period of corrosion-fatigue" has been entered. The second period begins when the fatigue limit of the corroded specimen has been reduced to equality with the "corrosion stress." If a tension test be used as a criterion of corrosion, however, it is possible unknowingly to enter the second period of corrosion-fatigue This Mr. Rawdon has actually done. Hence the progress of pitting in his experiments has been due to ordinary fatigue as well as to corrosion.

In his last paragraph Mr. Rawdon says that, if low frequency be used, the ratio of fatigue to corrosion is less than if high frequency be used. As just explained, however, ordinary fatigue should be avoided in investigating the effect of cyclic stress on corrosion. If this be done, cycle frequency has no such effect as Mr. Rawdon assumes in his last paragraph.

That Mr. Rawdon's experiments were continued far into the second period of corrosion-fatigue is evident from the fact that some of his specimens failed under cyclic stress. The same fact will be evident on comparison between the results that he obtained with duralumin and the results reported in a paper by the writer.[1] At first sight it may seem strange that more of his specimens did not fail under corrosion. The explanation, however, is that in Mr. Rawdon's alternating-bend test the bending moment decreases as the specimen yields, whereas in the rotating-cantilever tests used at Annapolis the bending moment remains practically constant throughout the test.

In Mr. Rawdon's machine the deflection remains constant throughout a test, hence as the specimen yields under cyclic stress and corrosion the bending moment decreases.

MR. H. S. RAWDON.[2]—Answering the specific question that Mr. McAdam raises concerning the machine used, I would say that in the machine we have used, we have a constant amplitude of flexure, no matter how much the specimen is corroded. It is bent through the same amplitude at the same rate until the specimen will actually part of its own accord if the test is carried far enough. Without question, there is a change in the stress concentration as the test proceeds. From the character of the attack as shown by the microstructure, however, one can be very sure that high concentrations of stress will be built up corresponding to the intercrystalline path followed by the attack. This is unavoidable, no matter what testing method is used.

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[1] D. J. McAdam, Jr., "Corrosion of Metals Under Cyclic Stress," see p. 250.

[2] Senior Metallurgist, U. S. Bureau of Standards, Washington, D. C.

FATIGUE RESISTANCE OF SOME ALUMINUM ALLOYS

BY J. B. JOHNSON[1] AND T. T. OBERG[2]

SYNOPSIS

This paper is presented as additional data on the endurance properties of aluminum alloys. Several investigators have presented such data, but a noticeable discrepancy exists in the results as reported. The endurance or fatigue limit has been assumed on the basis of about 300,000,000 cycles. The results reported in this paper indicate that 500,000,000 cycles are insufficient to determine the endurance limit. An endurance limit at 300,000,000 cycles is satisfactory for most practical purposes, but with modern aircraft equipment a propeller may be subjected to over one billion alternations of stress in a year's service. The results of this investigation have led to a reduction in the allowable stresses used in the design of aircraft propellers.

The actual fatigue limit, that is, the stress at which a metal will run indefinitely in a rotating-beam type machine, has never been determined for the aluminum alloys. Previous investigators have stopped their tests before failure and estimated the fatigue limit from a continuation of the curve. Endurance limits reported by previous investigators for alloys similar to those discussed in this paper are as follows:

ALLOY	INVESTIGATOR	ENDURANCE LIMIT, LB. PER SQ. IN.	NUMBER OF CYCLES
A	R. R. Moore[3]	14 000	400 000 000
A	R. L. Templin[4]	15 000	250 000 000
B	D. J. McAdam, Jr.[5]	16 000	100 000 000
C	R. L. Templin[4]	15 000	350 000 000[a]

[a] Specimen not broken.

The results given in this paper are still inconclusive, but carry the fatigue curve beyond the point determined by previous investigators. Heat-treated aluminum alloys subjected to a large number of alternations of stress at a stress above the endurance limit are permanently injured. The extent of the injury is a function of stress and number of cycles.

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[1] Chief, Material Section, Materiel Division, Air Corps, U. S. A., Wright Field, Dayton, Ohio.

[2] Materials Test Engineer, Materiel Division, Air Corps, U. S. A., Wright Field, Dayton, Ohio.

[3] R. R. Moore, "Resistance of Manganese Bronze, Duralumin, and Electron Metal to Alternating Stresses," *Proceedings*, Am. Soc. Testing Mats., Vol. 23, Part II, p. 106 (1923).

[4] R. L. Templin, "Discussion of Fatigue of Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 140 (1927).

[5] D. J. McAdam, Jr., "Corrosion-Fatigue of Non-Ferrous Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 102 (1927).

MATERIAL

The materials were a heat-treated and a commercial-annealed duralumin-type alloy (designated A) and a special high-silicon duralumin with and without magnesium (designated B and C, respectively). The chemical analyses and physical properties of the alloys are given in Table I. The specimens of alloys A and B were cut from $\frac{9}{16}$-in. round bar stock. The specimens of alloy C were cut from the hubs of propeller forgings and represent material from a forged bar approximately 4 in. in diameter.

TABLE I.—CHEMICAL ANALYSIS AND PHYSICAL PROPERTIES OF THE ALLOYS.

CHEMICAL COMPOSITION

Alloy		Copper, per cent	Silicon, per cent	Iron, per cent	Manganese, per cent	Magnesium, per cent	Aluminum, per cent
A	Heat Treated	4.01	0.22	0.45	0.73	0.57	Difference
	Annealed	3.94	0.16	0.72	0.67	0.58	Difference
B		3.90	1.24	0.33	0.39	0.57	Difference
C		4.42	0.82	0.54	0.83	Nil	Difference
		4.56	0.82	0.84	1.12	Nil	Difference

AVERAGE TENSILE PROPERTIES AND BRINELL HARDNESS

Alloy		Proportional Limit, lb. per sq. in.	Tensile Strength, lb. per sq. in.	Elongation in 2 in., per cent	Reduction of Area, per cent	Brinell Hardness (500-kg. Load)
A	Heat Treated	27 800	59 300	18.0	38.6	117
	Annealed	8 300	33 800	21.5	45.8	61
B		45 500	66 200	10.7	31.9	142
C	Specimen of axis parallel to flow lines of forging	27 000	58 500	16.3	29.6	113
	Specimen of axis perpendicular to flow lines of forging	31 100	61 100	14.1	17.6	...

METHOD OF PROCEDURE

The endurance tests were made on a plain bearing, rotating-beam type machine previously described by Mr. R. R. Moore.[1] The specimens were machined to the form and dimensions of the standard short specimen used in this machine. All specimens were given a final polish with magnesia powder suspended in a water solution. The manipulation of the polisher was in an axial direction so that all traces of radial cracks on the surface of the test specimen were eliminated. All specimens were either coated with mineral oil or with lanoline containing a red dye immediately after machining. Wherever specimens were retested at a higher stress than the original, the loads for the retest were applied after stopping the machine, but without removing the specimen from the machine.

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[1] R. R. Moore, "Effect of Corrosion upon the Fatigue Resistance of Thin Duralumin," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 128 (1927).

RESULTS AND DISCUSSION

The curves shown in Fig. 1 are drawn through average points, each point representing the average of several determinations for each stress except the points above 400,000,000 cycles, which represent the failure of only one or two specimens. For example, the point for a stress of 18,000 lb. per sq. in. is 20,390,000 cycles (for alloy C), which represents the average obtained on 12 test specimens. The lowest number of cycles obtained in computing this average was 11,803,400; the greatest number of cycles, 24,931,000. All points on the curves represent the actual failures at the original load. No points representing retests at higher loads are given on the curve sheet. The prolongation of the curve by a dotted line is simply to indicate that it is the opinion of the present investigators that the fatigue limit of the aluminum alloys is something less than indicated by the solid line, but that the point at which the curve becomes asymptotic is undetermined. In fact, there is no conclusive evidence to show that the curve ever becomes asymptotic.

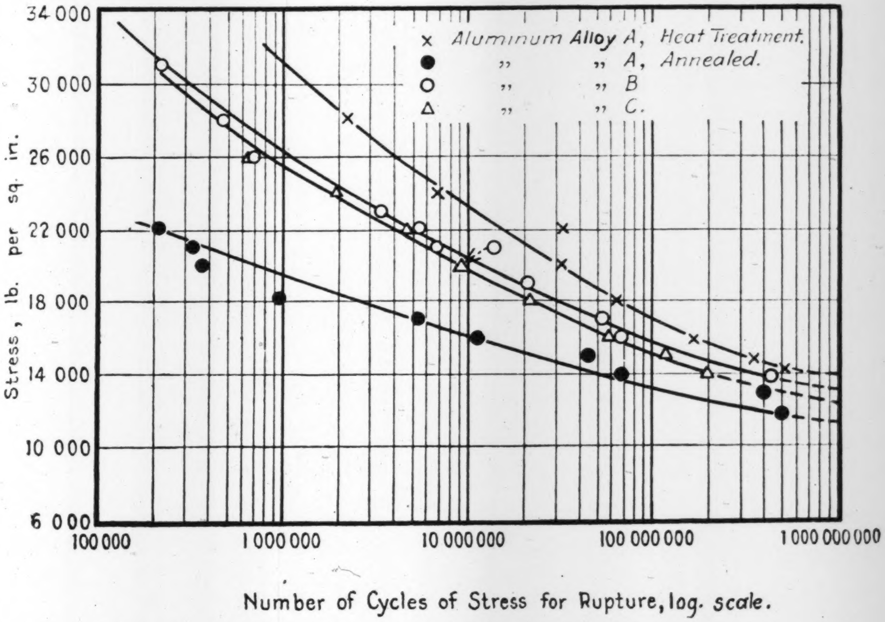

FIG. 1.—Relation of Stress to Number of Cycles of the Alloys.

Several of the test specimens were run to 500,000,000 cycles. In case the specimen had not failed at 500,000,000 cycles, the stress was increased and the specimen run to failure. Several specimens were also run at 100,000,000 cycles and at 200,000,000 cycles and then retested at higher stresses to determine whether the previous operation at low stress had injured the specimens. The results of restressing the heat-treated alloy A are given in Table II. The variation in the number of cycles on the retest after 100,000,000 cycles is not significant inasmuch as there was only one specimen for each stress. The results simply indicate that 100,000,000 cycles at a stress of 14,000 lb. per sq. in. or less do not injure or strengthen the specimen, as the number of cycles to failure compares favorably with 65,000,000, which is taken from the curve as representative of the number of cycles to failure of a specimen originally stressed at 18,000 lb. per sq. in. The specimen run to 400,000,000 cycles at 15,000 lb. per sq. in. was actually injured, as were the specimens stressed at 14,500 and 14,000 lb. per sq. in. and run to 500,000,000 cycles. There seems to be no doubt that, whereas no injury will result after

TABLE II.—FATIGUE PROPERTIES OF ALLOY A, HEAT TREATED.

Original Stress, lb. per sq. in.	Cycles to Failure from Curve	Average Number of Cycles at Which Specimen Failed at a Stress of 18,000 lb. per sq. in.		
		After Operating at Original Stress for 100,000,000 Cycles	After Operating at Original Stress for 400,000,000 Cycles	After Operating at Original Stress for 500,000,000 Cycles
15 000	300 000 000	......	16 000 000	......
14 500	450 000 000	......	......	9 000 000
14 000	700 000 000[a]	77 000 000	......	14 000 000
13 000	......	36 000 000	......	......
11 000	......	63 000 000	......	......
9 000	......	56 000 000	......	......

[a] Estimated on prolongation of curve.

TABLE III.—FATIGUE PROPERTIES OF ALLOY C.

Original Stress, lb. per sq. in.	Cycles to Failure from Curve	Average Number of Cycles at Which Specimen Failed at a Stress of 18,000 lb. per sq. in.	
		After Operating at Original Stress for 100,000,000 Cycles	After Operating at Original Stress for 200,000,000 Cycles
15 000	110 000 000	8 000 000	4 000 000
14 000	200 000 000	15 000 000	19 000 000
13 000	450 000 000[a]	12 000 000	11 000 000
12 000	......	27 000 000	18 000 000
11 000	......	27 000 000	20 000 000
10 000	......	27 000 000	......

[a] Estimated on prolongation of curve.

100,000,000 cycles at 14,000 lb. per sq. in., serious injury is apparent after 500,000,000 cycles.

The tests on the alloy C (Table III) are more conclusive, as the results are the average of several specimens. This alloy is injured after operating 100,000,000 cycles at 13,000 lb. per sq. in. and after operating 200,000,000 cycles at 12,000 lb. per sq. in. Tests are now being made at 11,000 and 10,000 lb. per sq. in. The stress on these will be increased after 400,000,000 and 500,000,000 cycles, respectively.

If a repeated low stress affects the material in such a way that an increase of stress will cause a failure below the normal fatigue curve for that material, the original low stress was above the fatigue limit; the low stress must have started a breakdown in the material which was accelerated by the application of a larger stress. This fact has a practical significance. There are cases where airplane propellers, after operating under normal service conditions, were tested on a test rig at stresses above those of normal operation and have failed at a fewer number of revolutions than similar propellers operated on the test rig that have never been in service.

DISCUSSION

MR. L. B. TUCKERMAN[1] (*presented in written form*).—The paper by J. B. Johnson and T. T. Oberg is valuable in adding considerable to the still meager data on the fatigue resistance of aluminum alloys.

It is interesting to see that they confirm again the conclusion of other investigators that even at 400,000,000 alternations of stress no endurance limit has been reached in these alloys. This emphasizes again the desirability of pursuing still more energetically the development of high-speed fatigue testing machines built on the resonance principle, such as the air-driven machine developed by L. J. Briggs at the U. S. Bureau of Standards, or the modification of it developed at the National Physical Laboratory in England.

If we expect our great-grandchildren really to know something definite about the endurance properties of aluminum alloys, something will have to be done now to hurry up the speed of testing.

MR. J. B. KOMMERS.[2]—The authors of this paper are to be complimented upon undertaking a task which they knew in advance would be a tedious one.

In 1920 to 1921, Mr. H. F. Moore and the speaker used a standard run of 100,000,000 cycles for testing a series of steels. These tests proved conclusively that an endurance limit actually existed for ferrous metals. It should be recalled that these tests were made only a few years ago.

Many tests since then have shown that unfortunately many of the non-ferrous metals must be tested to many more cycles than the ferrous metals in order that the endurance limit may be determined. The aluminum alloys, as has been shown, are particularly recalcitrant in this respect. If, however, we are to be able to substitute facts for guesses, it is necessary for someone to carry out actual tests.

A very interesting feature of the present paper are the results obtained to show the damage which is done by low stresses, and the relation of this damage to the number of applied cycles. I am very glad to know that this work will be continued.

In connection with alloy A, I wish to suggest to the authors that they include a statement as to the heat treatment and the annealing treatment used, so that the facts may be more complete.

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[1] Assistant Chief, Division of Mechanics and Sound, U. S. Bureau of Standards, Washington, D. C.

[2] Professor of Mechanics, Engineering College, University of Wisconsin, Madison, Wis.

MR. J. M. LESSELLS[1] (*presented in written form by Mr. R. E. Peterson*).—The paper by Messrs. Johnson and Oberg is interesting and brings some new data on fatigue of aluminum alloys. The most striking point is the continued falling off in the values of endurance limit as the number of cycles are increased. Evidently since this must have an influence on the application of aluminum alloys, it is to be hoped that more data will be forthcoming.

A further significant point is the fact that here we have tests running to 500,000,000 cycles. Surely this must bring to our attention once again the importance of developing a short-time test if this be at all practicable.

MR. R. L. TEMPLIN.[2]—The tests on the specimen reported as not broken at 350,000,000 cycles have since been carried out to approximately 500,000,000 cycles.

In 1925,[3] the speaker presented some similar data before this Society on the same subject, giving results of specimens that had been tested to two billion cycles of stress. At that time in our fatigue work, we found we were not paying enough attention to the protection of the specimens that were running those long periods of time. It should be appreciated that two billion cycles of stress means about two years and a half, day and night, at 1500 r.p.m. During such time, there is opportunity for many things to take place in the tests which have not been given proper consideration. This causes me to wonder if the authors of the present paper have given attention to all the details that should be considered in obtaining the results presented. In another paper presented before this annual meeting by Mr. Kommers,[4] we have the point emphasized that the test specimens must run true if the results are to be consistent. Trueness of the specimens might well be a governing factor in the results obtained from exceptionally long-time fatigue tests, but this is only one factor and the speaker is of the opinion that before we obtain the final answer in the fatigue tests of materials which require such long-time tests, much attention must be given to the other factors involved which are frequently neglected.

MR. D. J. MCADAM, JR.[5]—The authors have evidently done much careful experimental work on this subject. I do not agree, however, with their presentation and interpretation of the data.

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[1] Engineer in Charge, Mechanics Section, Research Laboratory, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

[2] Chief Engineer of Tests, Aluminum Co. of America, New Kensington, Pa.

[3] Discussion on Non-Ferrous Fatigue Tests, *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part II, p. 95 (1925).

[4] J. B. Kommers, "The Fatigue Properties of Cast Iron," see p. 100.

[5] Metallurgist, U. S. Naval Engineering Experiment Station, Annapolis, Md.

TABLE I.—CHEMICAL COMPOSITION OF ALUMINUM ALLOYS.

Material	Designation	Copper, per cent	Aluminum, per cent	Iron, per cent	Manganese, per cent	Magnesium, per cent	Silicon, per cent
R. R. MOORE							
Duralumin Rods.......	1923	3.25	[a]	0.28	0.70	Trace	0.28
Duralumin Tubes......	1927	4.26	94.26[b]	0.23	0.49	0.55	0.21
JOHNSON AND OBERG							
Aluminum Alloys......	A	4.01	[a]	0.45	0.73	0.57	0.22
Aluminum Alloys......	B	3 90	[a]	0.33	0.39	0.57	1.24
Aluminum Alloys......	C	4.42	[a]	0.54	0.83	Nil	0.82
Aluminum Alloys......	C	4.56	[a]	0.84	1.12	Nil	0.82
McADAM							
Aluminum Alloys......	DVA	4.55	93.25[b]	0.56	0.82	Nil	0.82
Aluminum Alloys......	DVB	4.60	93.25[b]	0.55	0.83	Nil	0.77
Aluminum Alloys......	GU	4.10	93.86[b]	0.57	0.55	0.63	0.29
Aluminum Alloys......	IN	3.86	94.01[b]	0.61	0.56	0.44	0.50
Aluminum Alloys......	DUA	4.28	93.80[b]	0.54	0.62	0.42	0.34
Aluminum Alloys......	GT	3.72	94.06[b]	0.42	0.50	0.28	1.02

[a] Difference.

[b] Obtained by difference.

TABLE II.—PHYSICAL PROPERTIES OF ALUMINUM ALLOYS.

Material	Condition	Designation	Tensile Strength, lb. per sq. in.	Johnson's Limit, lb. per sq. in.	Proof Stress, lb. per sq. in.	Elastic Limit, lb. per sq. in.	Proportional Limit, lb. per sq. in.	Elongation in 2 in., per cent	Reduction of Area, per cent	Charpy Impact Value, ft-lb.	Shearing Strength, lb. per sq. in.	Brinell Hardness (500-kg. Load)
R. R. MOORE												
Duralumin Rods....	Annealed	1923	25 250	.....	.....	6 230	6 800	25.0	60.8	....	.....	...
Duralumin Rods....	As Rec'd.	1923	50 960	.....	.....	22 500	25 030	16.0	50.4	....	.....	...
Duralumin Rods....	Tempered	1923	51 170	.....	.....	16 880	18 550	29.3	47.5	....	.....	...
Duralumin Tubes...	....	1927	69 400	.....	.....	18 000	40 365	16.8	....	....	.....	...
JOHNSON AND OBERG												
Aluminum Alloys...	Heat Treated	A	59 300	.....	.....	.....	27 800	18.0	38.6	....	.....	117
Aluminum Alloys...	....	B	66 200	.....	.....	.....	45 500	10.7	31.9	....	.....	...
Aluminum Alloys...	[a]	C	58 500	.....	.....	.....	27 000	16.3	29.6	....	.....	113
Aluminum Alloys...	[b]	C	61 100	.....	.....	.....	31 100	14.1	17.6	....	.....	142
Aluminum Alloys...	....	..	.....	.....	.....	.....	.....	....	....	....	.....	...
McADAM												
Aluminum Alloys...	Tempered	DVA	59 500	29 500	29 800	24 200	19 900	24.2	37.4	....	.....	...
Aluminum Alloys...	[c]	DVB	51 680	.....	19 000	.....	.....	....	43.5	....	.....	...
Aluminum Alloys...	Tempered	GU	54 600	25 100	25 600	20 900	15 900	24.5	36.9	7.6	37 600	...
Aluminum Alloys...	Tempered	IN	58 100	33 200	33 700	33 200	21 400	27.6	42.1	12.2	16 500	...
Aluminum Alloys...	Tempered	DUA	61 600	27 500	27 300	20 800	16 000	29.5	39.4	10.0	41 200	...
Aluminum Alloys...	Tempered	GT	69 100	45 300	45 500	40 700	36 800	17.8	22.8	....	.....	...

[a] Specimen of axis parallel to flow lines of forging.

[b] Specimen of axis perpendicular to flow lines of forging.

[c] Quenched, not artificially aged.

It is unfortunate that they did not publish results of individual tests, but merely plotted averages. Moreover, their method of averaging is not correct. As a semi-logarithmic scale was used in plotting results, it is incorrect to average the numbers of cycles. If averages are to be used, the logarithms of the numbers of cycles should be averaged. Because they averaged the numbers of cycles instead of the logarithms their plotted averages are too far to the right.

In the accompanying Fig. 2, however, I have used the authors' averages and have drawn interpretative curves. These are the first three graphs in the upper row of the figure. For comparison, results obtained in other laboratories have been assembled in the accompanying Figs. 1 and 2. The graphs have been redrawn to illustrate the writer's interpretation of the data. Chemical composition and physical properties are given in the accompanying Tables I and II.

In Fig. 1 are graphs representing results obtained at the Naval Engineering Experiment Station. Most of these results have been presented in various papers.[1,2,3,4] It will be observed that the stress-cycle relationship is best represented by a relatively broad curved band rather than by a single curved line. The curved lines in the figure represent approximate upper and lower boundaries of this band. These graphs illustrate the fact that results of fatigue tests of duralumin usually show a rather wide "scatter." Failure to realize this fact has caused much misinterpretation of the results of fatigue tests of duralumin.

These graphs also illustrate the fact that stress-cycle graphs for duralumin, when extended upward, often show a reversal of curvature. Failure to realize this fact has contributed somewhat to the misinterpretation of results of fatigue tests of duralumin.

The decrease in ordinate of the stress-cycle graphs in Fig. 1 between abscissas representing 10^7 and 10^8 cycles varies from about 2000 to 4000 lb. per sq. in., except for duralumin GU. For duralumin GU, the decrease is nearly 7000 lb. per sq. in. For that material, as shown in a paper[2] by the writer, there is an unusual relationship between the ordinary fatigue graph and the corrosion-fatigue graphs. Because of the abnormally long straight slope of the

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[1] D. J. McAdam, Jr., "Endurance Properties of Non-Ferrous Metals," Part II, Paper No. 1537-D. Issued with *Mining and Metallurgy*, February, 1926.

[2] D. J. McAdam, Jr., "Fatigue and Corrosion-Fatigue of Metals," Internat. Congress Testing Mats., Amsterdam, 1927.

[3] D. J. McAdam, Jr., "Corrosion-Fatigue of Non-Ferrous Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 102 (1927).

[4] D. J. McAdam, Jr., "Corrosion of Metals Under Cyclic Stress," see p. 250.

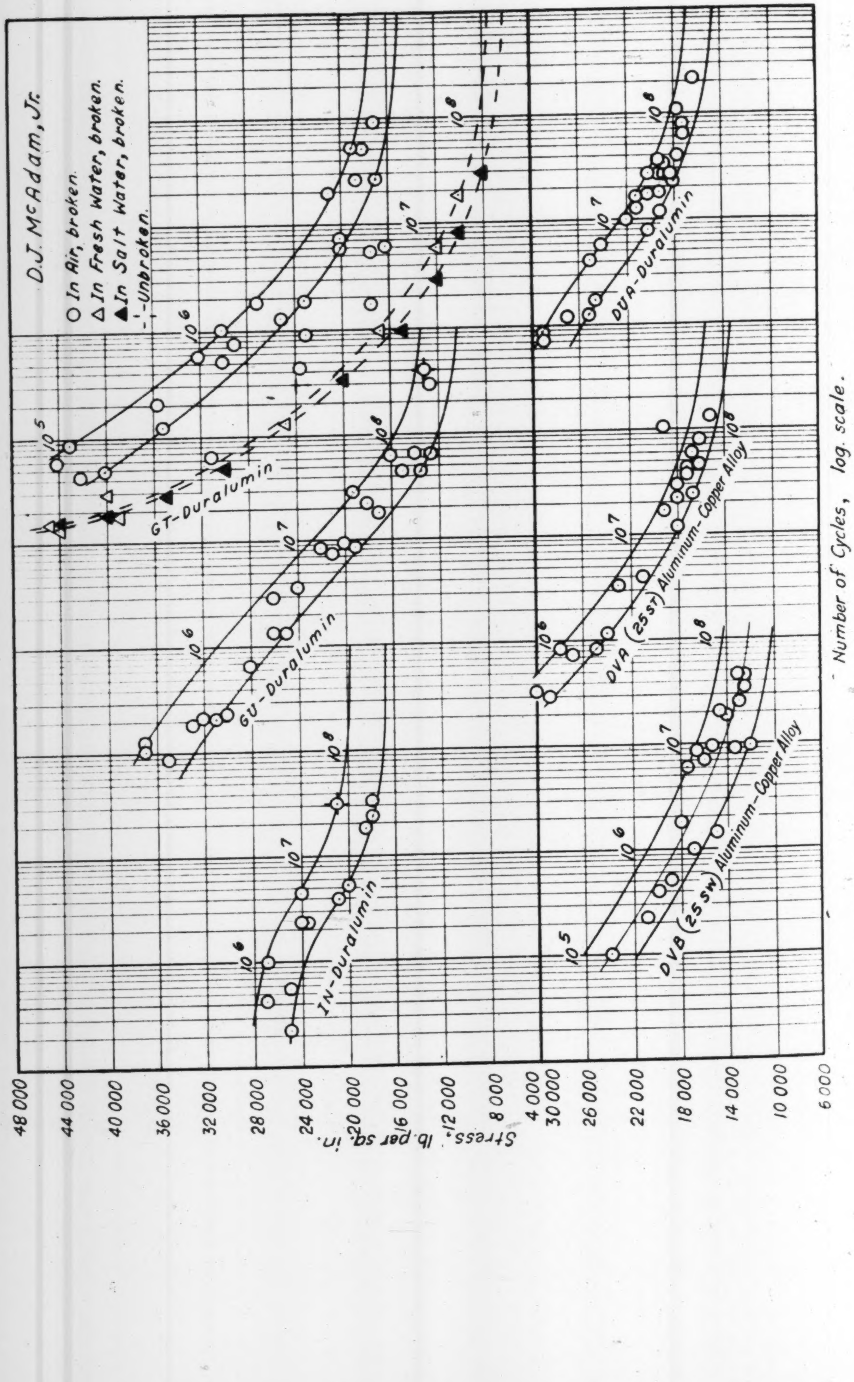

FIG. 1.—Results of Fatigue Tests on Aluminum Alloys Obtained at the Naval Engineering Experiment Station.

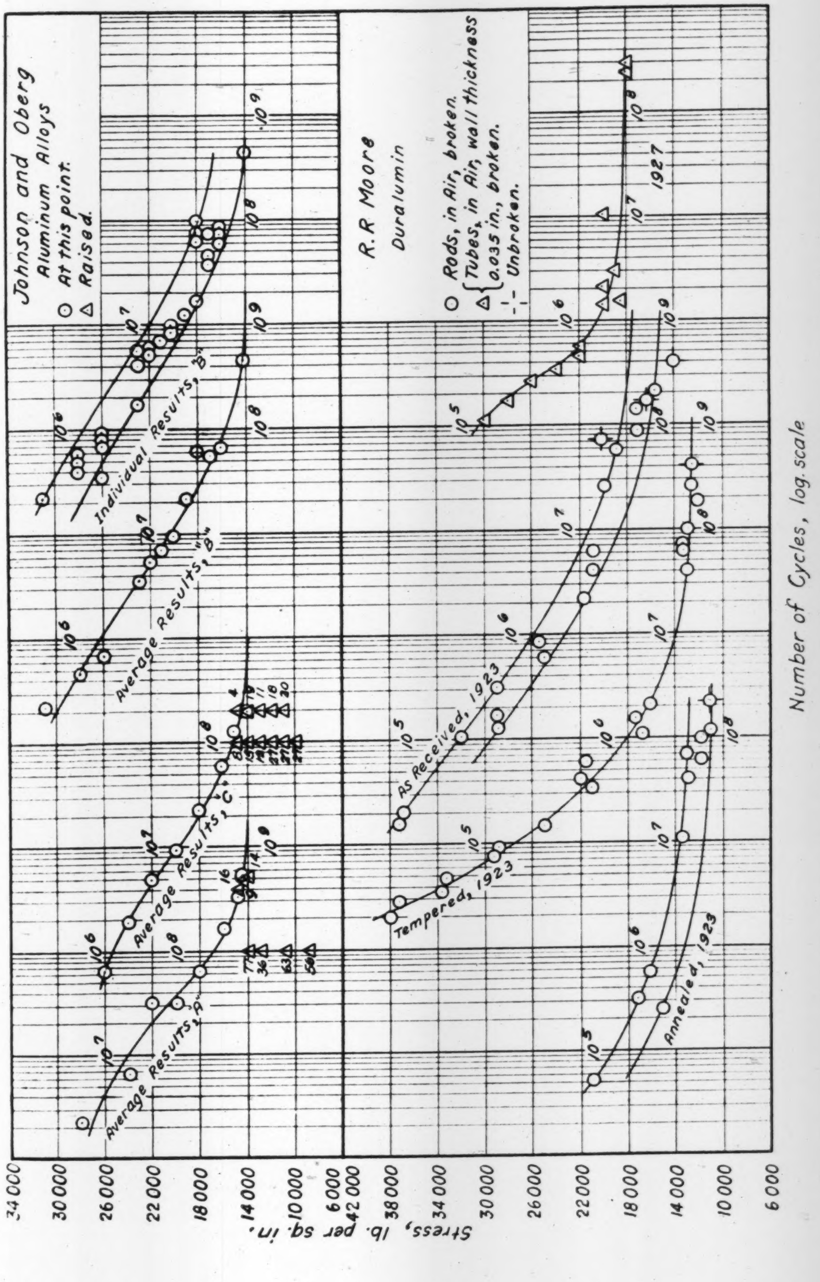

FIG. 2.—Results of Fatigue Tests on Aluminum Alloys Obtained from Data by Johnson and Oberg, and R. R. Moore.

ordinary fatigue graph the corrosion-fatigue graphs approach the ordinary fatigue graph at both the upper and lower ends. This surprising relationship has not been observed with any other duralumin sample. This relationship, as stated in the writer's paper,[1] seems to indicate that for this sample of duralumin, the long slope of the ordinary fatigue graph and the unusually low endurance limit are due to atmospheric corrosion-fatigue. It was also suggested in the same paper that atmospheric corrosion-fatigue may account for the long approach to a horizontal asymptote observed in most duralumin graphs.

Even for duralumin GU, however, the slope of the stress-cycle graph is evidently slight between abscissas representing 10^8 and 10^9 cycles. The slope of the graphs in Fig. 1 between abscissas 10^7 and 10^8 probably ranges from about 500 to 1500 lb. per sq. in. The corrosion-fatigue graphs as well as the ordinary fatigue graphs in Fig. 1 all seem to be approaching a horizontal asymptote.

In the upper row of the accompanying Fig. 2, interpretative graphs have been drawn to represent the results obtained by the authors. In the first three graphs the authors' averages have been used. It will be observed that these graphs have about the same form as the graphs presented in the accompanying Fig. 1. The graph on the right represents the results of individual tests kindly sent to the writer by Mr. Johnson. This graph resembles in form and scatter the graphs obtained at the Naval Experiment Station. For each of the four graphs representing the authors' results the indicated decrease of ordinate between abscissas 10^7 and 10^8 is about 1500 lb. per sq. in.

The lower row of graphs of the accompanying Fig. 2 have been redrawn from results presented in papers by R. R. Moore.[2,3] The three graphs on the left represent results of his first paper on this subject. The graphs, as redrawn with recognition of the relatively wide "scatter" obtained with most samples of duralumin, resemble closely in form the graphs obtained by the authors and the graphs obtained at the Naval Experiment Station. The indicated decrease in ordinate for the annealed material between abscissas 10^7 and 10^8 is only about 1000 lb. per sq. in. For the two hardened samples the indicated decrease in ordinate between abscissas 10^8 and 10^9 is only about 1000 lb. per sq. in.

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[1] D. J. McAdam, Jr., "Fatigue and Corrosion-Fatigue of Metals," Internat. Congress Testing Mats., Amsterdam, 1927.

[2] R. R. Moore, "Resistance of Manganese Bronze, Duralumin, and Electron Metal to Alternating Stresses," *Proceedings*, Am. Soc. Testing Mats., Vol. 23, Part II, p. 106 (1923).

[3] R. R. Moore, "Effect of Corrosion upon the Fatigue Resistance of Thin Duralumin," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 128 (1927).

The graph on the right of the lower row represents results obtained with duralumin tubing. The results indicate that for these specimens the decrease in ordinate between abscissas 10^7 and 10^8 is only about 1000 lb. per sq. in. and between abscissas 10^8 and 10^9 is negligible.

The results presented in Figs. 1 and 2, therefore, indicate that with some samples of duralumin the stress-cycle graph becomes practically horizontal between abscissas 10^7 and 10^8. With other samples of duralumin the decrease of the ordinate may continue appreciably beyond 10^8 cycles, but the decrease in the ordinate between abscissas 10^8 and 10^9 is not more than about 1500 lb. per sq. in. The abnormality of the graphs obtained with some samples is probably due to atmospheric corrosion-fatigue. The assembled evidence, however, indicates that in absence of atmospheric corrosion-fatigue, the stress-cycle diagram for duralumin is normal, and that duralumin has a definite endurance limit.

The assembled evidence also indicates that little is gained by extending endurance tests of duralumin beyond 100,000,000 cycles. In view of the scatter of results, it is better to use the available time in testing to not more than 100,000,000 cycles, so as to determine the form of the stress-cycle band. The endurance limit can then be obtained with sufficient accuracy by extrapolation.

MESSRS. J. B. JOHNSON[1] AND T. T. OBERG[2] (*authors' closure by letter*).—The authors wish to express their appreciation of the several criticisms of their paper. We are unable to state the exact heat treatment of alloy A, as suggested by Mr. Kommers, as these alloys were purchased in the condition in which they were tested and no data could be obtained other than that they were given the standard commercial heat treatment for this type of material.

With regard to the criticism of Mr. McAdam, it was realized by the authors that it would be mathematically more correct to plot from the averages of the logarithms, but the effect on the position and slope of the curve was negligible. It is noticeable that, if the average results "*B*" as replotted by Mr. McAdam are superimposed on the individual results "*B*" in Fig. 2, the curve comes half way between the upper and lower limits. The authors believe that Mr. McAdam has a tendency to give too much importance to one or two points in shaping his curves. It would take far more data than is at present available to warrant an assumption that there is a reversal of curvature in the fatigue curve for duralumin. Certainly, the authors' results do not warrant any such conclusions.

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[1] Chief, Material Section, Materiel Division, Air Corps, U. S. A., Wright Field, Dayton, Ohio.

[2] Materials Test Engineer, Materiel Division, Air Corps, U. S. A., Wright Field, Dayton, Ohio.

In presenting these data, the authors were primarily interested in the practical aspects of the fatigue properties of aluminum alloys. For some applications it may be entirely satisfactory to use an endurance limit at 100,000,000 cycles or to extrapolate the curve beyond that point. The determination of the allowable working stresses for aircraft parts cannot be made on a hypothetical basis. The endurance characteristics of materials are being used as a basis for design and must be determined accurately. The authors believe that their results indicate clearly that the aluminum alloys are weakened by the application of a low stress for several hundred million cycles. It is our opinion that no investigator has ever published sufficient data to warrant a statement that the fatigue curve for aluminum alloys can be extrapolated to indicate a fatigue limit below which the material will have an indefinite life.

FATIGUE STUDIES OF NON-FERROUS SHEET METALS

BY J. R. TOWNSEND[1] AND CHARLES H. GREENALL[1]

SYNOPSIS

The paper describes the development of a fatigue test machine for sheet metals and gives results of fatigue tests on five alloys of alpha brass, one alloy of nickel silver, one alloy of phosphor bronze and Everdur.

The results indicate that cold work raises the endurance limit but not proportionally to the increase in tensile strength produced by the same cause.

Micrographs are shown indicating that fatigue failure of the metals investigated is transcrystallin.

Dispersion hardening of alpha brass by nickel silicide increases the endurance limit.

The ratio of endurance limit to tensile strength of these alloys varies from 0.12 to 0.36, depending upon the composition, heat treatment, and cold work. These ratios are much lower than similar ratios for steel.

INTRODUCTION

The materials referred to in this paper are those non-ferrous sheet metals that are employed in electromechanical devices such as telephone apparatus, including a wide variety of equipment, for example, switches, relays, jacks, contact springs,[2] etc. These metals are employed mostly in springs used for electrical contacting purposes. In many cases these springs have precious metal contacts welded to them and in other cases the metal itself is used for the contact. Many of these springs are subjected to millions of cycles of stress, and it is important, therefore, that the endurance limit of these materials be known in order that apparatus may be designed which will endure for its required service life. Very little precedent has been established in the design of fatigue testing machines for the testing of sheet metals, and it was necessary, therefore, to develop a form of fatigue machine especially suitable for these materials.

Another paper presented at this annual meeting describes these non-ferrous metals and explains various methods of test and the commercial limits developed for specification purposes.[3]

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[1] Members Technical Staff, Bell Telephone Laboratories, Inc., New York City.

[2] John R. Townsend, "Telephone Apparatus Springs," *Transactions*, Am. Soc. Mechanical Engrs., 1928; also *The Bell System Technical Journal*, April, 1929, p. 257.

[3] J. R. Townsend, W. A. Straw and C. H. Davis, "Physical Properties and Methods of Test for Some Sheet Non-Ferrous Metals," see p. 381.

SHEET METAL FATIGUE SPECIMEN

The specimen shown in Fig. 1 was designed to simulate in its major dimensions the normal size of the springs used in telephone apparatus. It will be noted that the design of the specimen provides a section of uniform stress for $\frac{3}{8}$ in. at approximately $\frac{1}{2}$ in. from the clamped end of the specimen. This is accomplished by designing a cantilever beam that will have a uniform bending moment for part of its length. The dash lines indicate the shape of a beam of uniform bending moment; the solid lines show the final shape of the specimen and how the portion of uniform bending moment has been connected by means of fillets. This is essential in order to eliminate the possibility of the clamping stresses affecting the applied stress as would be the case if a uniform rectangular cantilever beam specimen had been employed. The specimen is deflected at a point $\frac{1}{2}$ in. from the operated end, or in other words, where the hypothetical beam of uniform bending moment terminates.

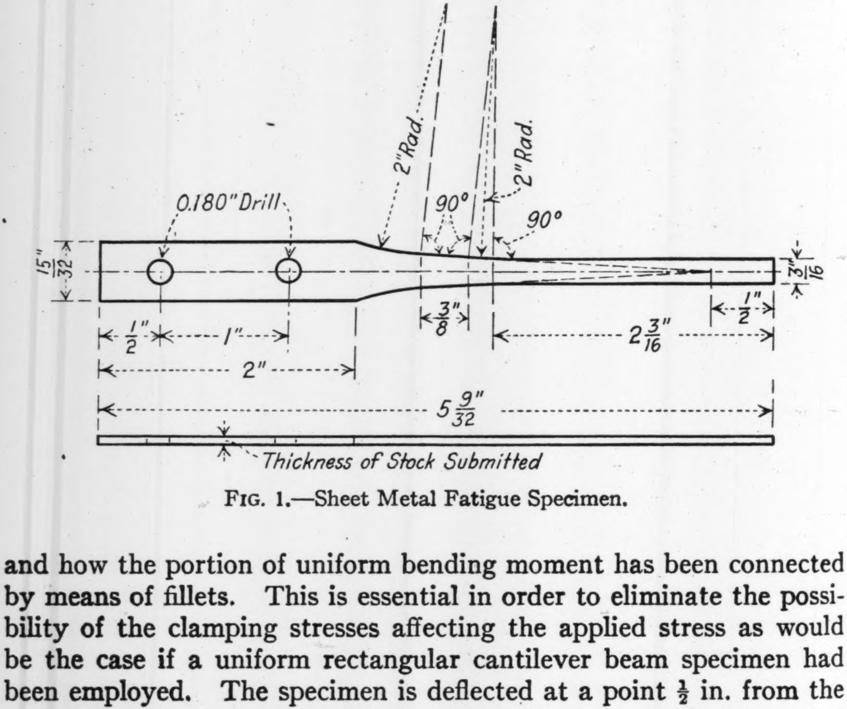

FIG. 1.—Sheet Metal Fatigue Specimen.

The specimens are prepared by blanking rectangular samples which are clamped together and then cross milled with a form milling cutter in a manner similar to that described elsewhere for the preparation of tensile specimens.[1] The specimens were cut with the direction of rolling parallel with their length.

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[1] R. L. Templin, "Methods for Determining the Tensile Properties of Thin Sheet Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 235 (1927).

SHEET METAL FATIGUE MACHINE

Referring now to Fig. 2, it is seen that the specimens *S* are clamped between phenol fiber blocks *B*. This is done in order that reasonably rigid material will be provided and at the same time a material sufficiently dissimilar to the metal to accomplish good clamping without scoring the surface of the clamped portion. Furthermore, the use of the phenol fiber blocks provides a means of automatically recording the breaking of the specimen since the specimen is insulated from the machine and may be employed to break an electrical monitoring circuit. The deflected end of this specimen is held between two fingers F_1 and F_2 which have a vertical cylindrical half section.

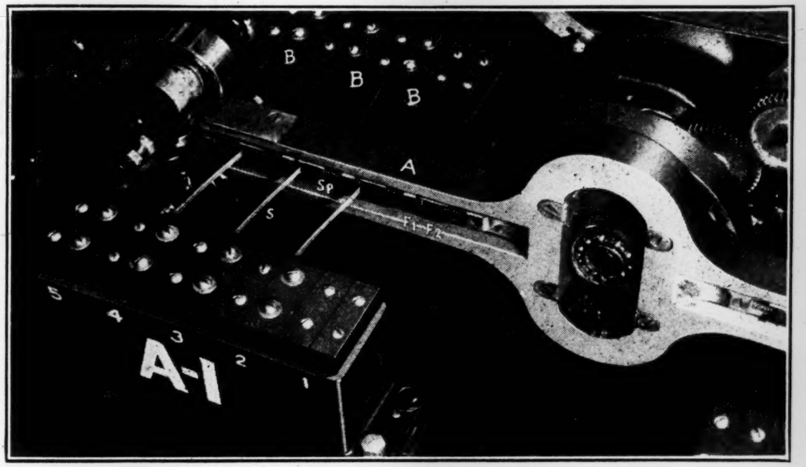

FIG. 2.—Sheet Metal Fatigue Machine.

The cylindrical portion is in contact with the specimen. This is necessary in order to compensate for the angular movement of the reciprocating arm *A* in relation to the specimen. One of these fingers, F_1, is fixed and the other, F_2, is movable but bears against the specimen with a pressure provided by compression spring S_p. This permits some slight movement of the specimen in relation to the fingers which is necessary by reason of the change in free length of the specimen as the reciprocating arm moves backward and forward. The deflection of the specimen is determined in two ways, first, by measuring the movement of the reciprocating bar and also by observing the specimen in operation by means of a stroboscope. In this way the static and dynamic deflection of the specimen may be measured, and for all practical purposes, these have been found to be the same within the range of deflection and speed used in this investigation.

The speed of the machine is approximately 1500 r.p.m. It is necessary to adjust the speed of the machine to the material under test, since if this is not done the machine may be operated near the natural frequency of vibration of the specimen and this may superimpose additional stresses upon it. This is determined by observing the operation of this specimen by means of a stroboscope and accurately setting the speed of the machine to a point where the vibratory motion of the specimen is uniform.

TABLE I.—AVERAGE CHEMICAL ANALYSIS OF MATERIALS.

Material[a]	Copper, per cent	Lead, per cent	Iron, per cent	Zinc, per cent	Nickel, per cent	Manganese, per cent	Tin, per cent	Phosphorus, per cent	Silicon, per cent	Combined Carbon, per cent
High Brass	65.09	0.02	0.03	Rem'der	....	....	....	....	....	....
Alloy G Brass	71.73	0.02	0.03	28.21	0.01	....	0.00	....	....	....
Nickel Silver	55.23	0.005	0.06	26.27	18.38	0.11	....	....	....	0.018
Phosphor Bronze	91.84	0.02	0.03	0.00	0.00	....	8.08	0.03	....	...
Everdur	95.46	....	....	....	....	0.99	....	....	3.55	....
Hardened Brass No. 33	Rem'der	....	....	9.89	2.32	....	....	....	0.57	....
Hardened Brass No. 34	Rem'der	....	....	19.89	2.37	....	....	....	0.57	....
Hardened Brass No. 35	Rem'der	....	....	30.12	2.36	....	....	....	0.66	....

[a] No graphite was present in any of the materials.

TABLE II.—PHYSICAL PROPERTIES.

Material	Heat Treatment	Temper, B. & S. Numbers Hard	Tensile Strength, lb. per sq. in.	Proportional Limit, lb. per sq. in.	Modulus of Elasticity, lb. per sq. in.	Elongation in 2 in., per cent	Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in. Ball, 100-kg. Load (Red Figures)	Endurance Limit, lb. per sq. in.	Ratio, Endurance Limit to Tensile Strength
High Brass	600° C. anneal	0	46 600	13 000	14 500 000	58	16	12 000	0.257
		4	77 200	32 000	....	6	79	13 500	0.175
		10	95 600	30 000	....	2	87	15 000	0.157
Alloy G Brass	600° C. anneal	0	46 300	.....	....	61	16	12 000	0.259
		4	81 600	......	....	6	84	18 000	0.220
		10	97 800	......	....	2	92	20 000	0.205
Nickel Silver	....	0	66 900	60 000	20 000 000	42	19	14 000	0.360
		4	98 700	......	....	2	69	18 500	0.174
		10	116 200	......	....	1.5	79	22 000	0.189
Phosphor Bronze	....	0	59 700	55 000	15 000 000	67	11	21 000	0.352
		4	95 500	......	....	14	71	22 000	0.230
		10	124 800	......	....	2	84	24 500	0.196
Everdur	Spring temper	..	80 000	26 000	12 400 000	22	91	24 000	0.300
Hardened Brass Alloy No. 33	Quenched 800° C., aged 1 hr. at 500° C.	..	90 000	44 500	19 800 000	14	86	14 000	0.155
Hardened Brass Alloy No. 34	Quenched 850° C., aged 1 hr. at 500° C.	..	85 800	37 200	17 200 000	21.5	85	12 500	0.146
Hardened Brass Alloy No. 35	Quenched 800° C., aged 1 hr. at 400° C.	..	85 400	38 000	16 500 000	28.0	79	16 000	0.173

The machine has a capacity of 40 specimens, 20 specimens being tested on each end of the motor drive. The machine is statically balanced and is smooth in operation. It is customary to test at least five specimens of each material at each stress. The machine therefore, has a capacity of 4 alloys of five specimens each at two deflections. The practice of using five specimens for each deflection was adopted because it was seen from the results of previous experimenters in fatigue testing that a more accurate result might be provided by doing so. The capacity of the machine is sufficiently large to permit this being done.

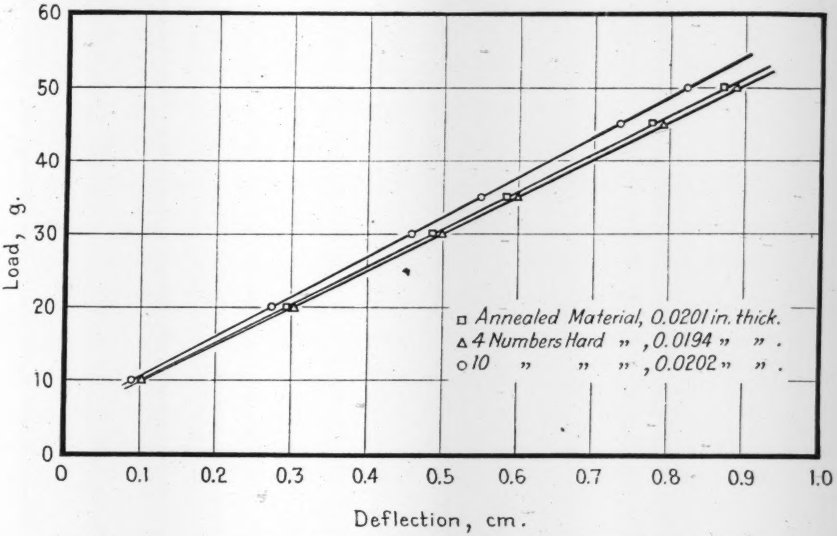

FIG. 3.—Relation of Load to Deflection, Alloy G Brass, No. 24 B. & S. Gage.

MATERIALS

The materials investigated consisted of five alloys of alpha brass, three of which had been hardened with nickel silicide, and one alloy each of nickel silver, phosphor bronze and Everdur. The three alloys of hardened brass have been described previously.[1]

The chemical composition of these alloys is given in Table I and the tensile strength, proportional limit, modulus of elasticity, percentage of elongation in 2 in., Rockwell hardness and endurance limit are given in Table II. The heat treatments and amount of cold work expressed by number of B. & S. gage reductions from standard anneal are shown also in Table II.

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[1] W. C. Ellis and Earle E. Schumacher, "Heat Treatment and Mechanical Properties of Some Copper-Zinc and Copper-Tin Alloys Containing Nickel and Silicon," *Proceedings*, Inst. Metals (1929).

FATIGUE ENDURANCE TEST

Method of Determining Stress in Specimen:

The method of determining the stress in the specimen consists in clamping a specimen in the same manner as on the fatigue machine. The clamped specimen is mounted on the table of a Société Genevoise Star Comparator so that its large surfaces are parallel with the vertical plane, the axis being parallel to the table. By mounting the specimen in this manner its weight has no appreciable effect on its deflection. The stress in the specimen is determined from the load deflection curve obtained by applying dead weights ½ in. from the end of the specimen and measuring the amount of deflection for

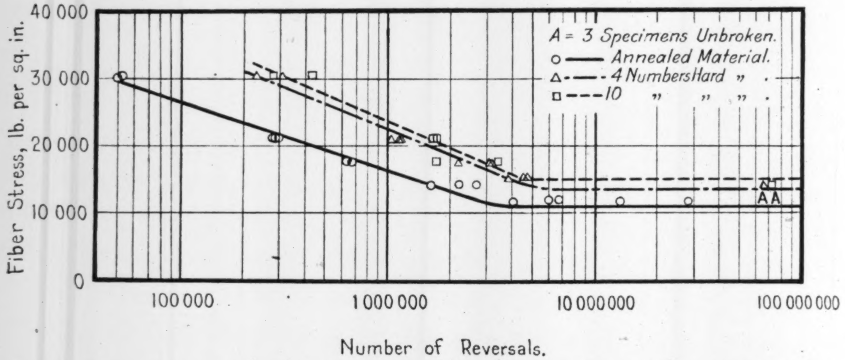


FIG. 4.—Relation of Fiber Stress to Reversals, High Brass Sheet, No. 24 B. & S. Gage.

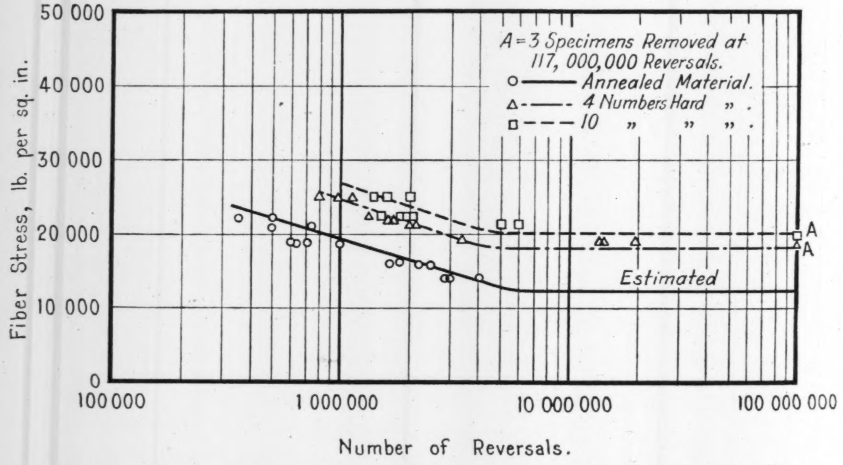


FIG. 5.—Relation of Fiber Stress to Reversals, Alloy G Brass Sheet, No. 24 B. & S. Gage.

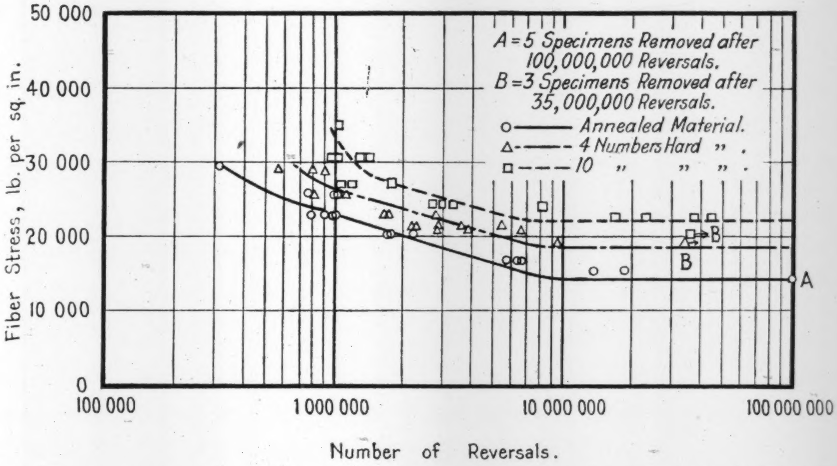


FIG. 6.—Relation of Fiber Stress to Reversals, Alloy B Nickel Silver, No. 20 B. & S. Gage.

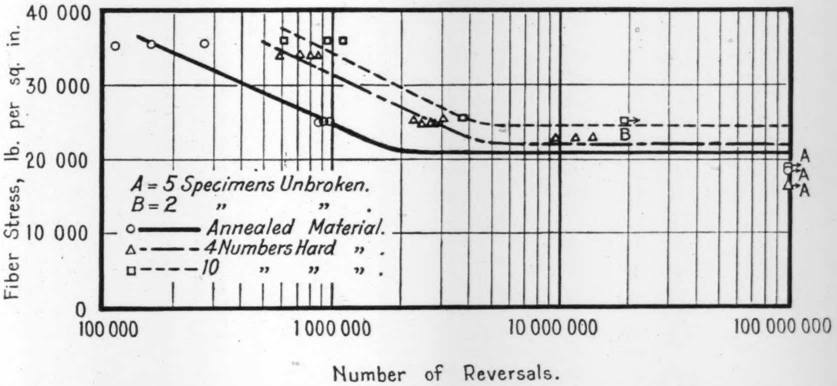


FIG. 7.—Relation of Fiber Stress to Reversals, Alloy C Phosphor Bronze, No. 24 B. & S. Gage.

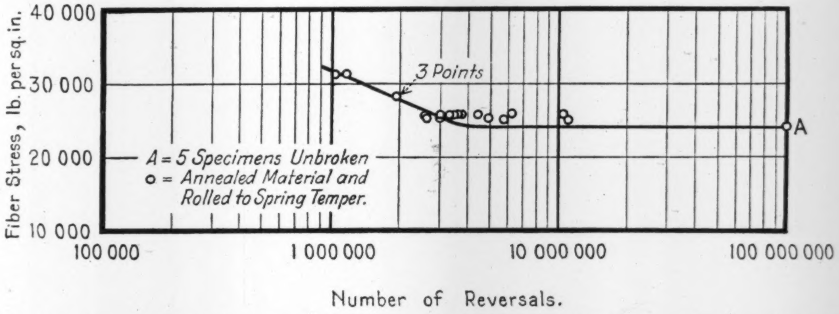


FIG. 8.—Relation of Fiber Stress to Reversals, Everdur Alloy Spring Temper, No. 24 B. & S. Gage.

various units of load by means of a microscope mounted on the comparator in such a manner that the rectangular coordinates of the deflected specimen may be read on the micrometer heads. Readings were taken to 1 micron. The stress per unit deflection is then calculated from the formula:

$$S = \frac{6Pl}{bd^2}$$

where S = stress in pounds per square inch,
P = load in pounds,
l = length in inches,
b = width in inches, and
d = thickness in inches.

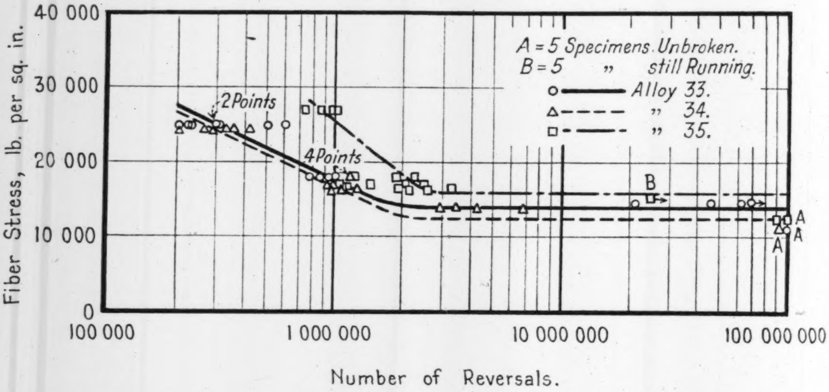


FIG. 9.—Relation of Fiber Stress to Reversals, Dispersion Hardened Brasses, Alloys Nos. 33, 34, and 35, No. 24 B. & S. Gage.

The curve shown in Fig. 3 for alloy G brass sheet gives the load for uniform deflection of the specimen upon the fatigue testing machine. The various stresses are then obtained by varying the amount of deflection of the end of the specimen by adjustment of the roller bearing that operates the reciprocating bar.

Fatigue Endurance Results:

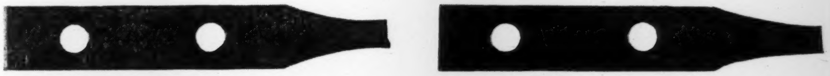
The curves shown in Fig. 4 are for high brass sheet annealed and rolled four and ten numbers hard. Figures 5, 6, 7 and 8 give similar results for alloy G brass, nickel silver, phosphor bronze and Everdur, respectively, for annealed material and rolled four and ten B. & S. gage numbers hard. Figure 9 gives the fatigue results for the alpha brasses hardened by nickel and silicon otherwise mentioned as alloys Nos. 33, 34 and 35, respectively.

MICROSTRUCTURE

Figure 10 shows a photograph of a number of broken specimens. The regularity of the break is shown and in practically every case occurs within the uniformly stressed area. Photomicrographs shown in Fig. 11 are typical of the various alloys. In these cases incipient cracks are revealed within the uniformly stressed area. It is seen that these cracks are, without exception, transcrystallin and there appears to be no distortion of the metal adjacent to the fractures. With regard to the typical structure of the hardened brass alloys reference is made to a previous paper.[1]

(*a*) Everdur alloy, spring temper. (*b*) Alloy C phosphor bronze, 4 numbers hard.

(*c*) Alpha brass alloy No. 33, heat treated. (*d*) Alpha brass alloy No. 34, heat treated.



(*e*) Alpha brass alloy No. 35, heat treated. (*f*) Alloy G brass, annealed.

(*g*) Alloy B nickel silver, annealed. (*h*) Alloy A brass, annealed.

FIG. 10.—Typical Broken Fatigue Specimen.

DISCUSSION

Examination of the results given in Table II reveals that the ratio of endurance limit to tensile strength for sheet non-ferrous metals is much lower than that reported for steel rod.[2] These ratios reported for plain carbon and alloy steels in all heat treatments vary from 0.35 to 0.67, averaging about 0.40, whereas for these sheet non-ferrous metals the ratios vary from 0.14 to 0.36.

The improvement in the endurance limit due to cold rolling brass, nickel silver and phosphor bronze is not consistent with the increase in tensile strength produced by the same means. This is

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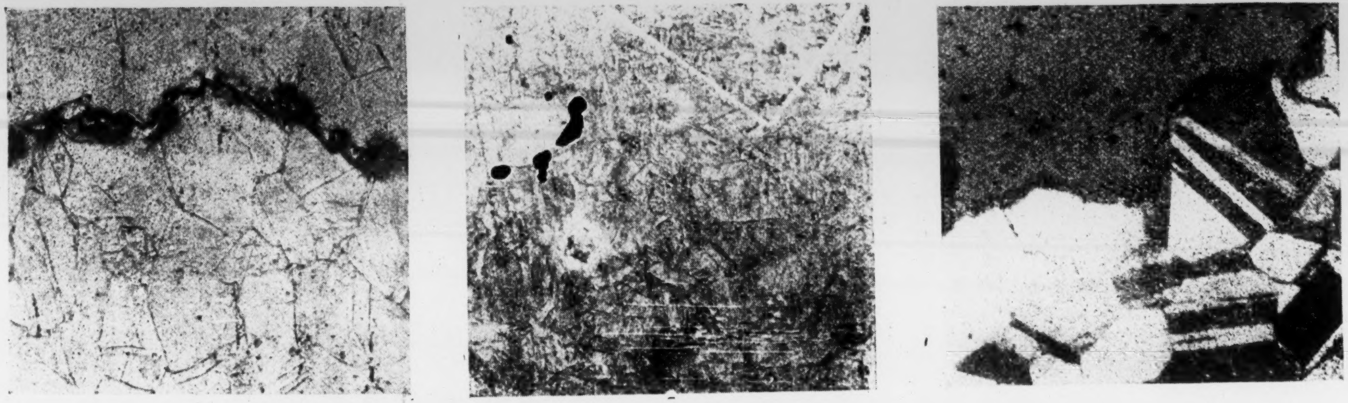
[1] W. C. Ellis and E. E. Schumacher, "Heat Treatment and Mechanical Properties of Some Copper-Zinc and Copper-Tin Alloys Containing Nickel and Silicon," *Proceedings*, Inst. Metals (1929).

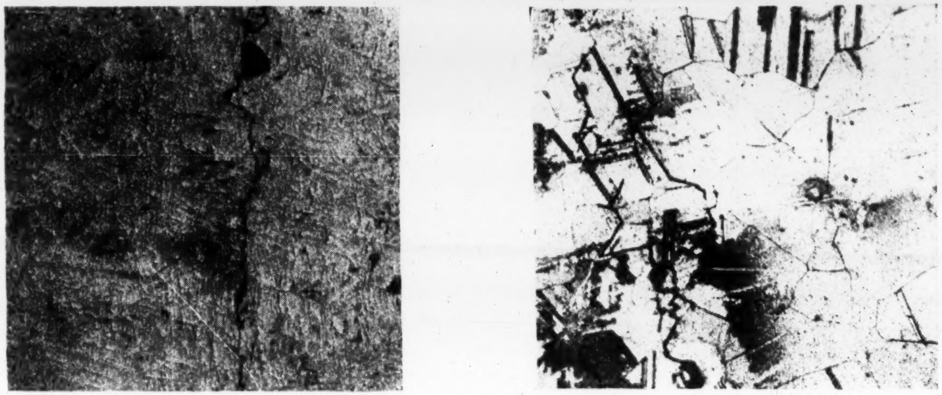
[2] H. J. Gough, "The Fatigue of Metals," Scott Greenwood and Sons; also H. F. Moore and J. B. Kommers, "The Fatigue of Metals," McGraw-Hill Book Co., Inc.



(*a*) Nickel silver, 4 B. & S. numbers hard, stress 28,750 lb. per sq. in., 800,000 cycles.

(*b*) Phosphor bronze, 4 B. & S. numbers hard, stress 255,000 lb. per sq. in., 2,600,000 cycles.

(*c*) Alloy G brass, annealed, stress 15,750 lb. per sq. in., 2,471,200 cycles.



(*d*) E verdur, spring hard, stress 25,500 lb. per sq. in., 1,186,900 cycles.

(*e*) Hardened brass alloy No. 33, stress 18,200 lb. per sq. in., 850,000 cycles.

FIG. 11.—Photomicrographs of Fatigue Specimens (× 200).

shown in Table II, where in practically every instance material hardened by cold work shows a progressive decrease in the ratio of endurance limit to tensile strength.

Previous investigators have shown a close correlation between endurance limit and tensile strength for iron and steel. Figure 12 fails to reveal such a general correlation for the sheet metals under test. For a particular alloy, however, there appears to be a progressive increase in endurance limit with increase in tensile strength due to cold work but the slopes of the curves shown are widely different for the various alloys.

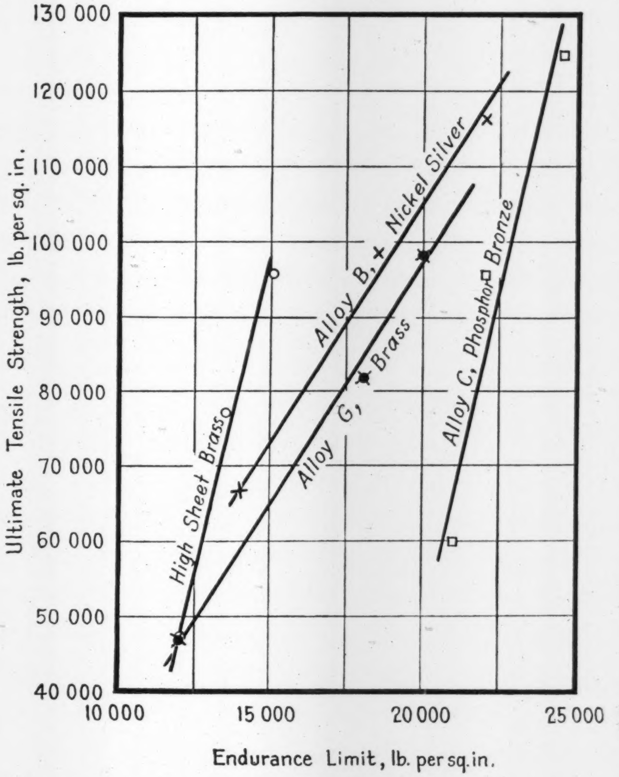


FIG. 12.—Relation of Tensile Strength to Endurance Limit, No. 24 B. & S. Gage, Non-Ferrous Sheet.

The results for alloy G brass and high brass show the effect of change in composition on the endurance limit. The greater improvement in fatigue endurance of alloy G brass due to cold work over high brass shows the superiority of this alloy for spring purposes. This was to be expected on the basis of its physical properties.[1]

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[1] W. H. Bassett and C. H. Davis, "Physical Characteristics of Copper and Zinc Alloys," *Proceedings*, Inst. Metals Division, Am. Inst. Mining and Metallurgical Engrs., p. 55 (1928).

Alpha brass hardened by nickel and silicon shows considerable improvement in the endurance limit and also an improvement in the ratio of endurance limit to tensile strength. This offers a means whereby some non-ferrous alloys may be improved in this respect.

Attention is called to Fig. 6 which gives the stress-cycle graphs for nickel silver in three tempers. It will be noted that the curves tend to turn sharply upward for the higher stresses. This indicates the effect of drastically overstressing the metal. The authors have observed this effect with other metals on the rotating beam machine. It seems that after a limiting stress value the number of cycles to failure tends to become constant.

CONCLUSION

From the test results obtained on these non-ferrous metals it is seen that the fatigue endurance limit varies from approximately 12 to 36 per cent of the tensile strength, whereas the commonly accepted ratio of fatigue endurance limit to tensile strength of steel is in the neighborhood of 40 per cent of its tensile strength. In other words, it appears that the low endurance limit of these materials emphasizes the need for their careful selection for use as springs. High tensile strength or proportional limit are not sufficient guarantors that the material will perform satisfactorily in service.

Cold work raises the endurance limit but not in a manner proportional to the increase in tensile strength produced by the same cause. There is no correlation between tensile strength and endurance limit except for cold-worked metal of a definite composition. For other compositions the correlation is different.

Precipitation hardening of alpha brass by nickel silicide increases the endurance limit.

The fatigue failure appears to be a result of a fracture across the crystals of the material. Photomicrographs are given showing incipient cracks that were developed in the uniformly stressed areas of the specimens.

The curves given for fatigue endurance show the results for each specimen tested. The shape of the fatigue endurance curve for these metals is similar to those published previously for other metals.

Acknowledgments:

Mr. L. E. Abbott assisted in the laboratory work. The photomicrographs were prepared by Miss A. K. Marshall. The high brass, alloy G brass, nickel silver, phosphor bronze and Everdur sheet were furnished by the American Brass Co. and Mr. C. H. Davis of that company provided their chemical analyses.

DISCUSSION

MR. L. B. TUCKERMAN[1] (*presented in written form*).—Messrs. Townsend and Greenall are to be congratulated on their notable contribution to the fatigue testing of thin sheet metals. Mr. Whittemore and I found it a sufficiently difficult problem to devise a machine which would satisfactorily test under known alternating stresses a single specimen of thin sheet metal, so that we have a keen appreciation of the amount of skill, ingenuity and patience which were necessary in order to make it possible for the authors to say of a machine testing 40 specimens at once, "In this way the static and dynamic deflection of the specimen may be measured, and for all practical purposes, these have been found to be the same within the range of deflection and speed used in this investigation."

The development of these 40-specimen machines increases very materially our prospects of securing adequate fatigue results on nonferrous alloys, within a reasonable length of time.

I should like to make one minor criticism. With no test results beyond 100,000,000 alternations of stress, I am wondering whether the stress-alternation curves beyond 10,000,000 alternations are with certainty known to be such beautifully horizontal straight lines. With duralumin breaks have been obtained well beyond 100,000,000 alternations.

MR. J. L. CHRISTIE[2] (*presented in written form*).—Is it customary to design telephone parts so that in normal use their true fatigue limits are not exceeded? If so, what factor of safety is used? I ask because a great many electrical devices are designed so as not to fail at say 50,000 cycles, or 100,000, and specifications call for minimum values as low as 6000. Material failing at 100,000 cycles in tests is considered perfectly satisfactory.

It is general practice to stress various members in these devices to well over their true endurance limits.

MR. J. M. LESSELLS[3] (*presented in written form by Mr. R. E. Peterson*).—The paper describes a new development in fatigue testing where departure is made from the conventional form of testing, and a constant deflection is maintained instead of a constant load. A similar development has been in progress at the Westinghouse Research Laboratory for thin strips using a somewhat similar arrangement.

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[1] Assistant Chief, Division of Mechanics and Sound, U. S. Bureau of Standards, Washington, D. C.

[2] Metallurgist, Bridgeport Brass Co., Bridgeport, Conn.

[3] Engineer in Charge, Mechanics Section, Research Laboratory, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

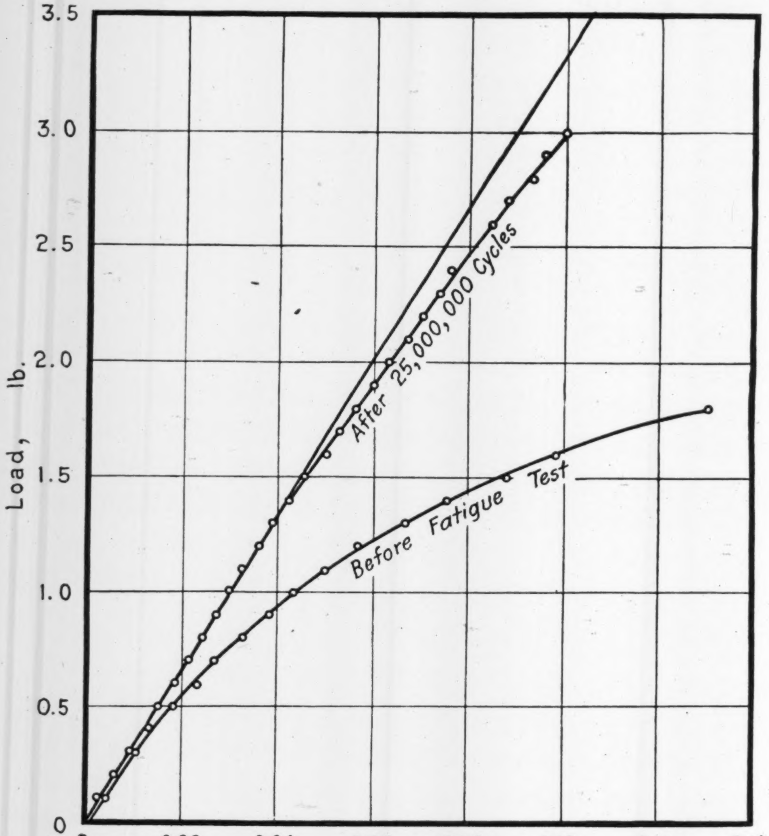

FIG. 1.—Stress-Strain Curves. Copper 10 per cent, cold rolled, annealed at 250° C. for 125 hours.

Some hesitancy has occurred as to publishing results until more work could be done. It was found that after subjecting the specimen to several million cycles the load to deflect a particular grade of specimen a given amount was very much greater than before the test, as shown in the accompanying Fig. 1. This means that we have the material subjected to a gradually changing stress which will cause large errors in endurance limit values. Whether this is true for all materials, the writer has not been able so far to determine.

MR. T. McL. JASPER[1] (*presented in written form*).—I should like to present an idea with reference to the question of whether the fatigue fracture of metals is essentially across crystal grains or around them. It seems to me that we have to consider the question of the strength of grain boundaries as compared to crystal strength. We have evidence which is fairly conclusive that the crystal is the first to solidify from the molten state. The boundary material the last to solidify. It is, therefore, evident that as the temperature increases in testing metal there must be a point reached in which the longer path around the crystal grain (I use this word with caution) is not so strong as the shorter path across the crystal grain. In low-carbon steel this temperature is believed to be in the neighborhood of 700 to 800° F. In steel in which the freezing zone is narrow this point may be different.

It is believed for some non-ferrous metals that the point mentioned above is below the ordinary service or testing temperatures.

To determine whether a metal has failed by fatigue on the basis of fracture across or around grains therefore may lead to a wrong conclusion. Since we are using in airplane engine service non-ferrous metals at elevated temperatures, the idea that fatigue fracture is transcrystalline should be used with caution. I do not believe that all metals fracture across the grain in fatigue even at ordinary temperatures.

MR. J. B. KOMMERS[2] (*presented in written form*).—I wish to call special attention to the first four metals listed in Table II of the paper, which were each tested in three degrees of hardness. These four metals exhibit properties which have been observed before by other investigators. When the tensile strength is increased in non-ferrous metals by cold work, the endurance limit is increased also, but by a much smaller percentage.

The maximum increase in tensile strength for the high brass was 105 per cent while the corresponding increase in endurance limit was only 25 per cent. For alloy G brass the corresponding figures are 111 per cent and 67 per cent. For the nickel-silver the figures are 74 per cent and 57 per cent, and for the phosphor bronze they are 109 per cent and 17 per cent. With the exception of the nickel-silver all the other metals show an increase in endurance limit which is only a small proportion of the increase in tensile strength.

In a paper before the American Society for Steel Treating in

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[1] Director of Research, A. O. Smith Corp., Milwaukee, Wis.

[2] Professor of Mechanics, Engineering College, University of Wisconsin, Madison, Wis.

1925,[1] McAdam showed that one of the reasons for these results on cold-worked material is the presence of internal stresses due to the cold-working. Experimenters both in England and in this country have found that these internal stresses may be greatly reduced by a so-called "low temperature anneal." The effect of the low temperature anneal is to relieve the internal stresses without decreasing appreciably either the hardness or the tensile strength. The effect obtained depends upon a combination of temperature and time of annealing. A temperature in the neighborhood of 500° F. was found to be very satisfactory, but it is evident that each metal and each degree of cold work probably has some particular combination of temperature and time which will produce the optimum effect.

MR. D. J. MCADAM, JR.[2]—In view of Mr. Kommers' reference to a previous paper, I should like to say that I would not apply to non-ferrous metals in general the statement that the endurance limit cannot be raised in proportion to the increase in tensile strength by cold working. We found for nickel-copper alloys that the endurance limit does come up in proportion to the increase in tensile strength by cold working, but I agree with Mr. Kommers that one would need to take care that internal stress is not present. We found with the copper-zinc alloys—which is in agreement with Mr. Townsend's paper—that even if we used a low temperature anneal to try to relieve internal stresses, the increase in endurance limit was not in proportion to the increase in tensile strength by cold working, though with copper we found that one can actually increase the endurance limit more than in proportion to the increase in tensile strength by cold working, so one cannot make a general statement in respect to non-ferrous metal.

I should like to call attention to one statement in the conclusions in which the authors give the ratio of endurance limit to tensile strength of steel as 40 per cent. I think that is a little low. I should put it nearer 45 per cent; for alloy steels it is nearer 50 per cent.

THE CHAIRMAN (*Past-President H. F. Moore*).[3]—May the Chair be permitted a word in connection with Mr. McAdam's discussion? Tests of cold-drawn copper at the University of Illinois do not show any increase of endurance limit over the value for annealed copper. Tests at Annapolis do show such an increase. These two conflicting results are before us at the present time. Possibly differences in internal stress in the two lots of cold-drawn copper may account for some of the difference, and possibly some of the effects discussed in the paper by Phillips and Edmunds[1] may be the cause of the different results.

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[1] D. J. McAdam, Jr., "Endurance Properties of Alloys of Nickel and of Copper," *Transactions*, Am. Soc. Steel Treating, Vol. VII, pp. 54, 217, 581 (1925).

[2] Metallurgist, U. S. Naval Engineering Experiment Station, Annapolis, Md.

[3] Professor of Engineering Materials, University of Illinois, Urbana, Ill.

MESSRS. J. R. TOWNSEND[2] and C. H. GREENALL[2] (*authors' closure by letter*).—With regard to the discussion by L. B. Tuckerman, we consider that 100,000,000 alternations of stress are sufficient to determine the fatigue life of the materials covered by this paper in view of the service to which these materials are to be exposed. We agree that it would be incorrect to conclude that the endurance limit for sheet non-ferrous metals could be determined absolutely on the basis of 100,000,000 alternations of stress. The data show, however, that there is very little change in slope of the curve for the various metals after 50,000,000 alternations of stress and it is reasonable to assume, therefore, that the results carried to 100,000,000 alternations of stress are reasonably close to the endurance limit for practical considerations. The same question could be raised with regard to any arbitrary number of alternations of stress since it is not possible to report results for an infinite number of alternations of stress.

With regard to Mr. Christie's comments on design practice with regard to endurance tests we attempt to keep the spring materials stressed below the endurance limit. Telephone design imposes severe restrictions on space in many instances and this in turn provides insufficient space for the proper size of spring members. In such instances it may be necessary to use material which may fail at the working stress after say 100,000 cycles where the service requirements may call for a total of 50,000 cycles. Instances of this kind are rare, however, and substitutions of material of higher endurance limits is usually resorted to in these circumstances. As a general proposition if the stress conditions of service are known there is no reason why the endurance limit may not be exceeded provided failure will not occur within the normal expected life of the part. As our knowledge of endurance testing and service stresses is improved, it will be possible to design stressed members to closely approach endurance values. Indeed, we will be forced to this in many circumstances on account of space limitations.

Mr. Lessells has raised a very interesting question regarding the possibility of sheet materials tested in the manner described in the paper developing greater stiffness after being on the fatigue tests some time. In the previous paper referred to above, mention was made that fatigue tends to cause the precipitation of elements held in a metastable condition. The authors would like to advance the thought that precipitation hardening might be responsible for the phenomena recorded by Mr. Lessells. In other words, fatigue will accelerate the age hardening tendencies of alloys and the increase in stiffness of the cold-rolled copper may be due to precipitation hardening by the impurities in the copper. The authors feel that these phenomena should be further investigated.

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[1] Arthur Phillips and Gerald Edmunds, "An X-ray Study of Copper Which Showed Directional Properties on Cupping," see p. 438.

[2] Member Technical Staff, Bell Telephone Laboratories, Inc., New York City.

Mr. Jasper has raised an interesting question about the relation of the path of the fatigue fracture as dependent upon temperature. It is true that at room temperature the path of the fatigue fracture in the materials covered by this paper is transcrystalline. In a previous paper by one of the authors[1] on "Fatigue Studies of Telephone Cable Sheath Alloys," it was shown that for pure lead and 1 per cent antimony lead cable sheath alloys the path of the fracture was intercrystalline. This would tend to bear out the contention of Mr. Jasper because it is known that these materials are not stable at room temperature. At a lower temperature the grain boundary weakness in these materials is improved whereas on the other hand as the temperture is raised the grain boundary weakness is correspondingly increased. It is agreed that fatigue tests carried on at normal temperatures cannot be applied with surety to conditions of service where the metals would be used either at a greatly raised or lowered temperature.

With regard to Mr. Kommers' remarks, the authors believe that the reversal of stresses produced by the fatigue tests would serve to reduce the internal stresses in the sheet material due to cold working. The authors have not experimented with materials in which the internal stresses have been reduced by low-temperature annealing. We believe that Mr. McAdams' and Professor Moore's comments are interesting in this connection.

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[1] J. R. Townsend, "Fatigue Studies of Telephone Cable Sheath Alloys," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 153 (1927).

FATIGUE TESTS OF LARGE SPECIMENS

By R. E. Peterson[1]

Synopsis

This paper deals with the development of large fatigue testing equipment of a rotating cantilever type. The purpose of larger equipment is to obtain results of greater uniformity for non-uniform material such as in weld metal or castings, and to test model rotors and other specimens having stress-concentration effects due to the presence of fine machined detail.

Three machines are described and the essential features of each pointed out.

Certain tests on welds are discussed from the standpoint of their bearing on the development of large machines.

Introduction

In connection with fatigue tests of specimens having discontinuities, it has been thought for some time that the use of larger specimens might be conducive to more uniform test results. The discontinuities may be in the material itself, such as in castings or in weld metal; or the discontinuities may be imposed in the form of fillets, holes, scratches, etc. It is well established from analytical and experimental investigations that the stress concentration caused by a discontinuity depends on the ratio of the size of the discontinuity to the size of the cross-section of the specimen. The effect of a blow hole $\frac{1}{32}$ in. in diameter would not be the same in specimens $\frac{1}{4}$ in. in diameter and 2 in. in diameter. The question of size effect is invariably raised by engineers in the application of fatigue test data to design. A further reason for the development of large fatigue testing equipment was the possibility of special model rotor tests. Such tests necessarily involve specimens of considerable fine detail which cannot be machined in specimens of such small size as to be tested in an ordinary fatigue machine.

Description of Apparatus

The rotating cantilever principle was found to lead to simpler and more economical designs of large equipment than a number of other principles of operation which were considered and was therefore selected as a basis of this development.

The first of the large machines developed is shown in Figs. 1 and 2. The specimen diameter is 1 in. at the critical section. It is apparent from the figures that the specimen also forms the drive shaft of the testing machine. This is made possible by using self-aligning ball bearings, which allow the shaft to deflect without introducing a binding action. Strictly speaking, the specimen thereby becomes an overhanging simple beam and not a cantilever beam, as in machines of this general type. A tapered sleeve feature furnished as standard equipment with such bearings for line-shaft application makes the changing of specimens a rather simple procedure and also

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[1] Mechanics Section, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

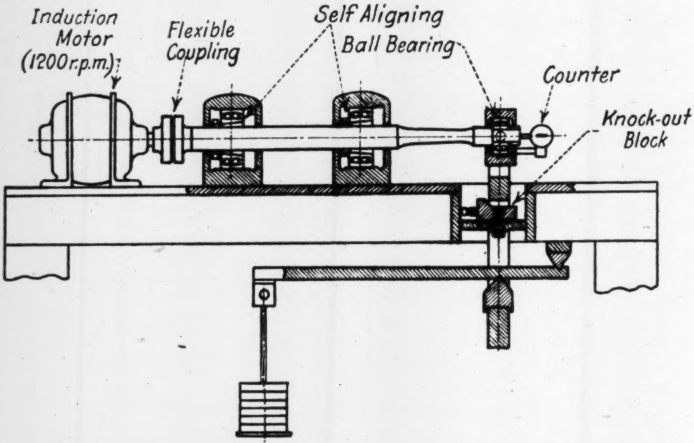

Induction Motor (1200 r.p.m.)
Flexible Coupling
Self Aligning Ball Bearing
Counter
Knock-out Block

FIG. 1.—Cantilever Fatigue Machine for Specimen 1 in. in Diameter.

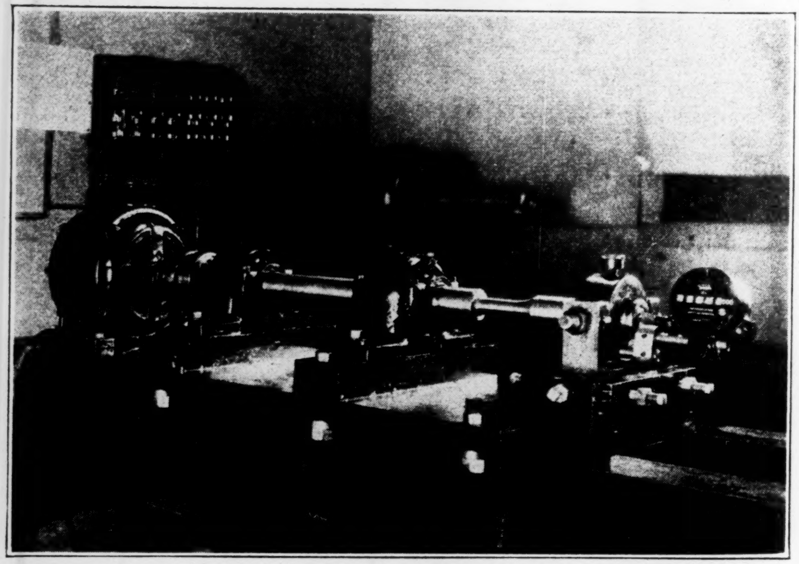


FIG. 2.—Photographic View of Cantilever Fatigue Machine for Specimen 1 in. in Diameter.

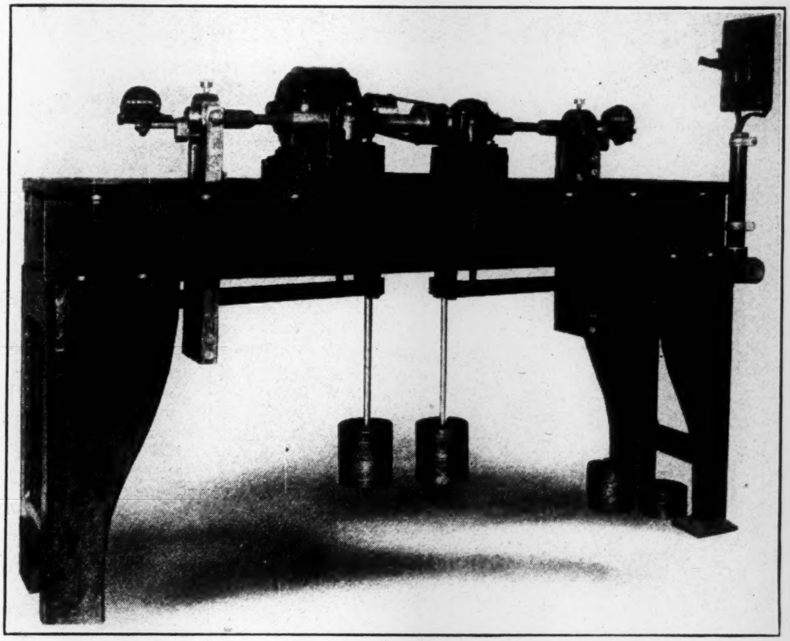


FIG. 3.—Double-Ended Cantilever Fatigue Machine for Specimens 1 in. in Diameter.

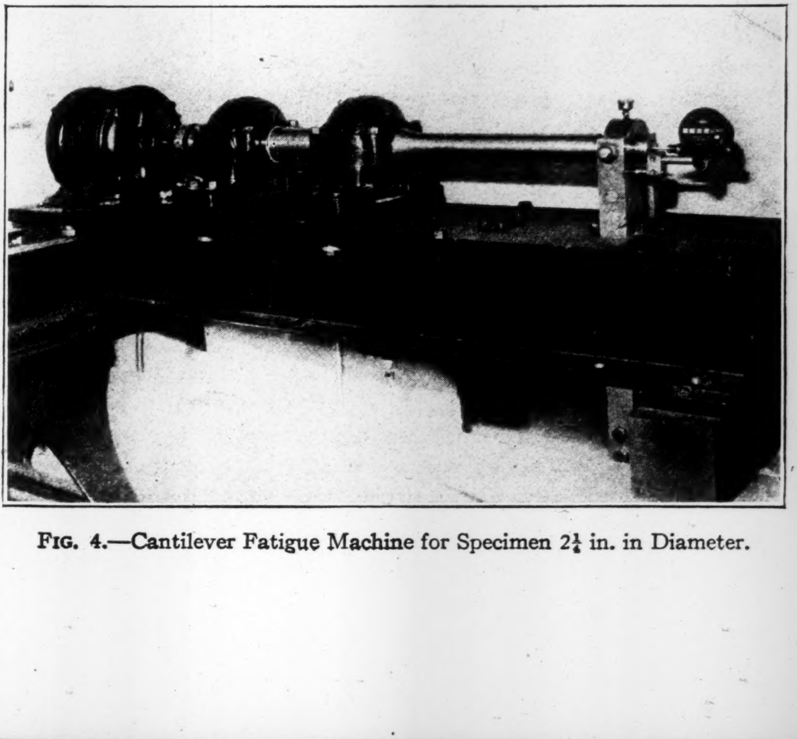


FIG. 4.—Cantilever Fatigue Machine for Specimen 2¼ in. in Diameter.

makes it unnecessary to hold the shaft portion of the specimen to close machining limits. The additional material required for the shaft end is not a serious objection, since the cost of the material for a fatigue specimen is usually but a small fraction of total cost of the complete specimen and the testing of it, particularly for ordinary steels.

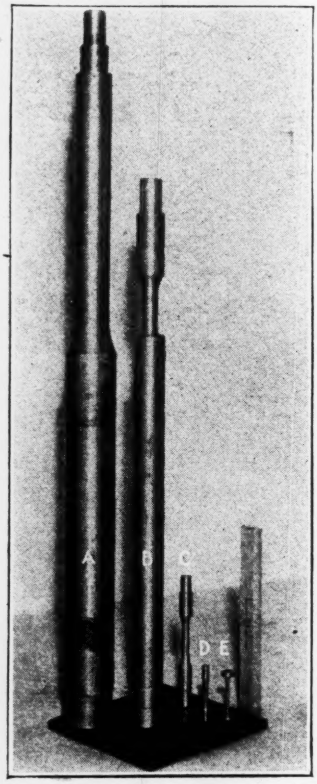


FIG. 5.—Comparative Size of Various Fatigue Specimens.

A, cantilever, $2\frac{1}{4}$ in. in diameter critical section.
B, cantilever, 1 in. in diameter critical section.
C, cantilever (McAdam type), 0.4687 in. in diameter critical section.
D, beam (Modified Farmer type), 0.300 in. in diameter critical section.
E, direct stress (Haigh type), 0.196 in. in diameter critical section.

The load is applied by means of the lever system shown in Fig. 1, the leverage being 4 to 1 for the machine shown. The weight of the lever system applies to the end of the specimen a definite force of considerable magnitude which must be determined before making any tests and which must be added thereafter in all tests to the effective force of the weights put on the loading pan. The knife-edges of the leverage system are of a swivel design, thereby allowing self-adjustment to take care of misalignment and loading distortion.

The machine shown in Figs. 1 and 2 is driven by a 1200 r. p. m. induction motor, direct-connected by means of a flexible coupling. All ball bearings are totally enclosed which keeps out dust and also allows thorough lubrication, both features being conducive to longer bearing life.

If the final complete fracture is of a jagged nature, the tendency sometimes is for the specimen not to pull completely apart but to run while slowing down in a manner similar to a toothed coupling, with considerable pounding and injury of the fractured surfaces. To obviate this condition, a "knock-out block" was designed as shown in Fig. 1 so that when the free end of the cantilever drops, it also is given an outward motion which disengages the broken surfaces.

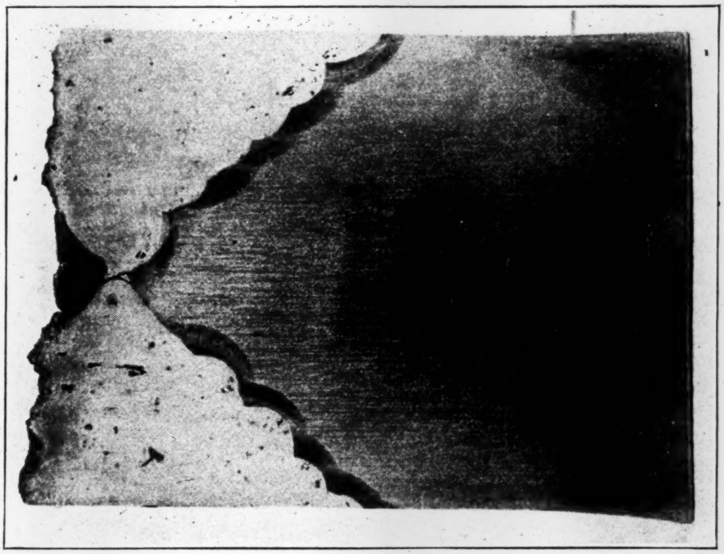


FIG. 6.—Fractured Section of Specimen $2\frac{1}{4}$ in. in Diameter, 45-deg. Double-Cone Weld.

The next machine developed (Fig. 3) is quite similar to the one just described, except that it is double-ended, thereby reducing the time required to obtain an endurance limit. The machine is belt driven, the driven pulley being on the shaft of the specimen. The specimen is also 1 in. in diameter at the critical section.

For model rotor tests a specimen $2\frac{1}{4}$ in. in diameter was needed for a model one-twelfth the size of an actual rotor, so a single-ended cantilever machine (Fig. 4) was developed for this work. The relative size of the specimens for the large machines as compared with standard specimens is shown in Fig. 5. The machine for specimens $2\frac{1}{4}$ in. in diameter was completed far ahead of the models because of difficulties encountered in making the models, so that the machine has been used for testing welded specimens. The $2\frac{1}{4}$-in. diameter is of advantage in welded conical surfaces such as shown in Fig. 6, which is a photograph of a macro-etched section (plane through axis) of a fractured specimen. Such a weld is made by progressing around the joint, which cannot be done successfully in small specimens.

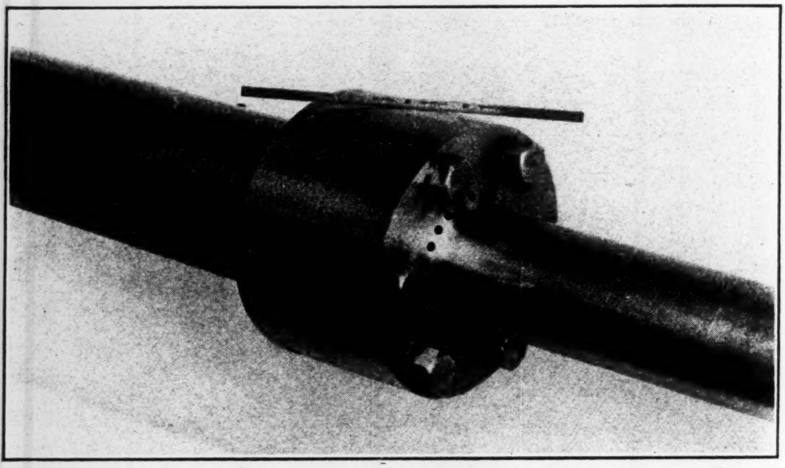


FIG. 7.—Model Rotor Fatigue Test Specimen.

The details of the machine illustrated in Fig. 4 are similar to those of the first machine described, except for an increase in size. The ball bearing at the free end of the cantilever was changed to a roller bearing in the larger machine, inasmuch as an effective force of approximately one ton is applied in certain tests. The leverage of the load application mechanism was increased from 4 to 1 to 7 to 1. If a load of one ton is applied at the free end of the cantilever, a force of $2\frac{1}{2}$ tons results at the middle bearing. Such a force applied to a bearing operating at 1200 r. p. m. causes considerable heat to be generated. The simple expedient of attaching a fan to the shaft near the bearing was found to provide sufficient air current to keep the bearing from heating excessively. It was thought that when the $2\frac{1}{4}$-in. specimen fractured some violent action might occur so several safety precautions were taken in the first tests. The failure proved to be quite peaceful, however, so no unusual safety precautions are now considered necessary. A model rotor test specimen representing at one-twelfth size a turbo-rotor shaft end is shown in Fig. 7. Fatigue tests will be made to determine the stress-concentration effect at the ventilation holes shown in the photograph.

TEST RESULTS

It is not the purpose of this paper to give a complete report of an investigation of the fatigue of welds, but only to present such phases of the investigation as have had a bearing on the development of large testing equipment. In the introduction it was stated that more uniform test results of specimens having discontinuities might be expected by using larger test specimens. Such discontinuities in weld metal and their effect in developing a fatigue crack are shown in Fig. 8. Some data pertaining to the question of uniformity of test results may be obtained from Figs. 9 and 10. Fatigue tests of the same type of weld (45-deg. double-V butt weld) in specimens 0.3 in. in diameter (Farmer-type machine) and 1 in. in diameter (cantilever machine, Fig. 2) are shown in Fig. 9. The difference in "scatter" is quite obvious. Fatigue tests of fillet welds are shown in Fig. 10. The "scatter" is not great, if the ragged nature of the unmachined fillet welds is considered. Photographs of macro-etched sections (plane through axis) of fractured specimens of the two types of fillet weld are shown in Fig. 11. It will be noticed that in the specimen shown in Fig. 11 (*a*), fracture occurred through the parent metal, the crack starting in the sharp corner at the base of the weld; while in Fig. 11 (*b*), fracture occurred entirely in the weld metal. In the former type of fracture, the average size of weld (dimension of leg of triangular cross-section of fillet) was 0.4 in. and in the latter type 0.3 in. This indicates that in this case a fillet weld of $\frac{3}{8}$ in. is the proper size, any substantial

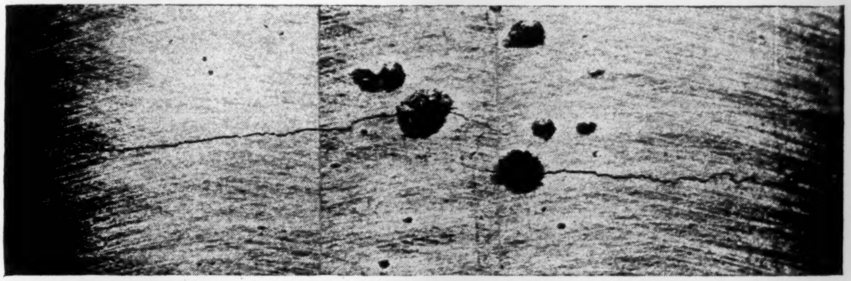


FIG. 8.—Fatigue Crack in Weld Metal (5,200,000 cycles, complete fracture at 5,270,000 cycles) (approximately ×7; originally photographed approximately ×13, reduced in reproduction).

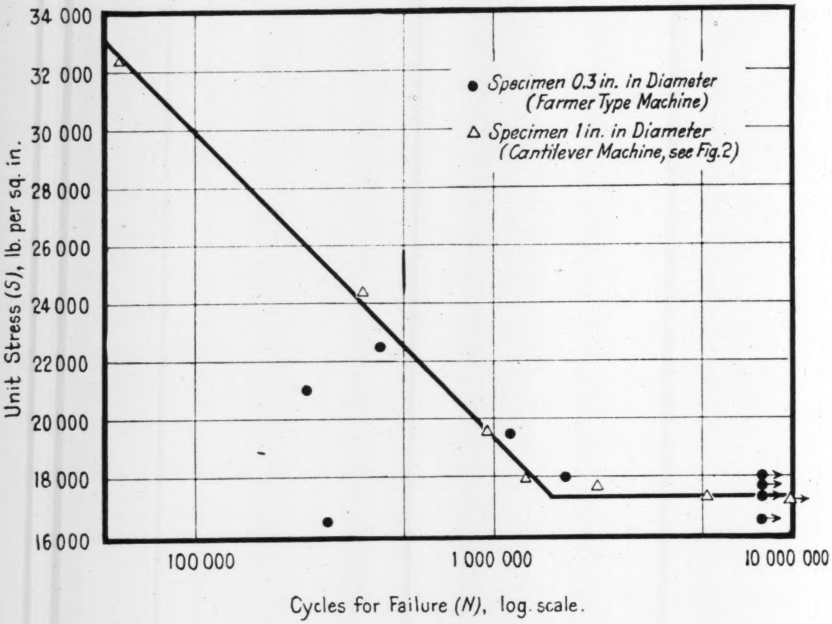


FIG. 9.—$S-N$ Diagram 45 deg. Double-V Butt Welds.

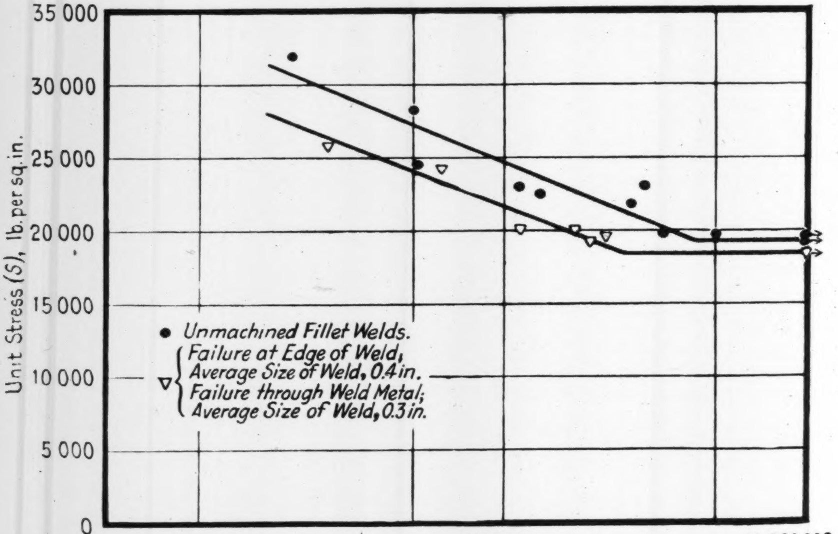
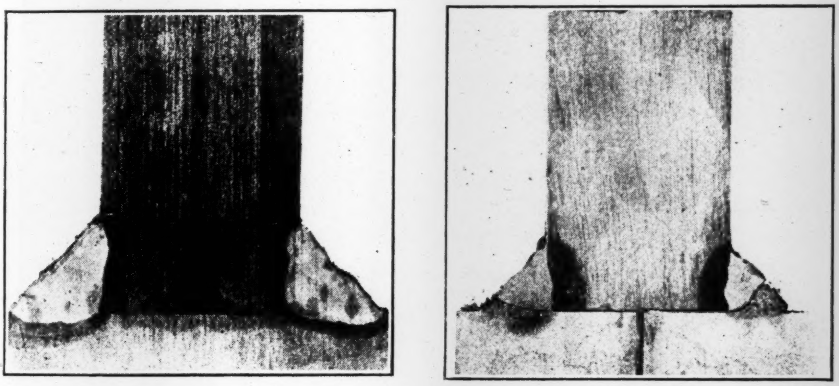


FIG. 10.—$S-N$ Diagrams for Fillet Welds (1-in. Cantilever Machine, see Fig. 2).

increase in size being a waste of weld metal and any decrease weakening the joint. It is realized that no generalization can be made of the above values, inasmuch as the shape and dimensions of the cross-section enter into the problem. Tests are now in progress to obtain further data of a fundamental nature on welded joints. The results of these tests will be published when the work reaches such a stage that definite conclusions may be drawn.

CONCLUSIONS

1. Fatigue tests of non-uniform material, such as weld metal, show less "scatter" if large specimens are used than if similar tests are made on specimens of small diameter.



(a) Failure through parent metal (size of weld, 0.4 in.)

(b) Failure through weld metal (Size of weld, 0.3 in.)

FIG. 11.—Fillet Weld Fractures.

2. Specimens of large diameter make it possible to test models and machine elements having the fine detail machining.

3. The fatigue strength of certain welded joints, such as fillet welds, can be investigated if specimens are 1 in. in diameter or larger. There is a definite minimum amount of weld metal necessary in the case of a fillet weld to cause failure through the parent metal.

Acknowledgments.—The author has been assisted by Messrs. W. G. Lundquist, W. L. Cory and R. S. Trinnaman in the operation of the machines. The photographs of the macro-etched sections were taken by Miss Mildred Ferguson. Particular acknowledgment is due Mr. G. Lobo, Jr., formerly research engineer on welding and Mr. J. M. Lessells, head of Mechanics Section, in the development of the large equipment.

DISCUSSION

MR. H. F. MOORE.[1]—At the University of Illinois the largest rotating-beam fatigue machine takes a specimen 2 in. in diameter. One end of the specimen is held in a conical chuck, and it has been found necessary to make that chuck with a liner of hardened steel in order not to wear out very rapidly. Case-carburized and heat-treated metal has not so far proved satisfactory.

Mr. Peterson's Fig. 9, while not conclusive, throws some light on certain phenomena met with in testing welds. The diagram for a small specimen is about the kind of diagram we frequently get for small specimens of welded material. While the tests of large specimens show a much smoother $S-N$ diagram, the lowest value of stress at which specimens broke is not very different for the two sets of tests. It may be that while large-specimen tests for welded material give better $S-N$ diagrams than small-specimen tests the endurance limit determined may be about the same.

MR. D. J. McADAM, JR.[2]—In a paper last year[3] I described experiments on corrosion-fatigue of large specimens. Slightly higher corrosion-fatigue limits were obtained with large than with small specimens. Mr. Lessells in discussing the paper suggested that size of specimen may also have some effect on the endurance limit obtained with uncorroded specimens. I should like to ask whether Mr. Peterson has investigated that problem. I suppose he has it in mind.

MR. R. E. PETERSON.[4]—The question of size effect raised by Mr. McAdam is on our test program but no data have been obtained as yet.

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[1] Professor of Engineering Materials, University of Illinois, Urbana, Ill.

[2] Metallurgist, U. S. Naval Engineering Experiment Station, Annapolis, Md.

[3] D. J. McAdam, Jr., "Some Factors Involved in Corrosion and Corrosion-Fatigue of Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 117 (1928).

[4] Mechanics Section, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

PHYSICAL PROPERTIES AND METHODS OF TEST FOR SOME SHEET NON-FERROUS METALS

By J. R. Townsend,[1] W. A. Straw[2] and C. H. Davis[3]

Synopsis

This paper continues an investigation undertaken to secure a simple and reliable method of test for sheet non-ferrous metals and the development of commercial test limits. A previous paper[4] presented data to show that the Rockwell hardness tester was the most suitable instrument for preliminary inspection purposes. Commercial hardness limits have been developed based on a Rockwell hardness - tensile strength relationship that has been determined for each alloy. In practice the Rockwell hardness limits are applied, but if the results are near to or outside the limits a tension test is made.

The Rockwell hardness and tensile strength limits are given for four alloys of brass, and two alloys each of nickel silver and phosphor bronze. The physical properties of the rolling series upon which these limits were based are presented as well as experience data obtained on shipments of commercial material. The limits for brass alloys A, B and E are considered final, but the limits for the other materials are tentative until more complete experience is available.

Grain size limits are given for annealed brass, nickel-silver and phosphor-bronze sheet. The grain count is made by comparison with the standard photomicrographs reproduced in the 1929 Report of Committee E-4 on Metallography.[5]

Refinements in the calibration of the Rockwell tester by using standard test blocks are given, as well as further development of testing technique. An experimental model of a motor-driven bend test machine is described.

Introduction

In a previous paper,[4] the need for accurate physical tests on sheet non-ferrous metals was emphasized and a review of the various methods of tests available was given. The results of the study of the methods of test suitable for high and clock brass sheet of the order of 0.020 in. and thicker were presented and it was shown that the

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[1] Member Technical Staff, Bell Telephone Laboratories, Inc., New York City.

[2] Development Engineer, Western Electric Co., Inc., Chicago, Ill.

[3] Assistant Metallurgist, American Brass Co., Waterbury, Conn.

[4] H. N. Van Deusen, L. I. Shaw and C. H. Davis, "Physical Properties and Methods of Test for Sheet Brass," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 173 (1927).

[5] *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part I, p. 505 (1929).

Rockwell tester gave the most reliable information of the several hardness testing machines available at that time. Methods of operation of the Rockwell hardness tester were worked out by the cooperating laboratories and a method of carrying on tension tests was developed. The limitations in the use of the Rockwell hardness tester for sheet metal were pointed out and the need for the development of a tester capable of higher precision in the harder materials and also capable of making hardness tests on material less than 0.020 in. thick was emphasized. The other methods of test investigated and described were the Scleroscope, Brinell, the Erichsen ductility and the bend test.

The tensile strength limits were determined by the following procedure. The tension test results of the rolling series were plotted against the actual percentage of reduction by cold rolling. Two limiting curves were drawn in giving the minimum and maximum tensile strength for all reductions covering the range of commercial anneals. These two curves were based on years of experience with these metals and also on the rolling series. The tensile strength limits were taken from these curves for the theoretically correct reduction for each temper. Rockwell hardness tests were made on the grip ends of the tension test specimens, thus establishing the Rockwell hardness - tensile strength relationship for the material. Having established the tensile strength - reduction relationship, the corresponding hardness values were obtained from the Rockwell hardness - tensile strength curve. These limits were then subjected to trial on a large number of shipments of material.

Considerable additional material has now been investigated and some slight modifications have been made in the Rockwell hardness limits. These changes are minor but tend to strengthen the agreement between Rockwell hardness and tensile strength for these materials. As explained in the previous paper the Rockwell test is considered a preliminary inspection test and is mainly useful because of its economy of time and material. The tension test, on the other hand, is considered the test upon which the acceptance or rejection of the material is based. In practice, material within the Rockwell hardness limits is accepted unless the hardness reading is near or outside the hardness limits, in which case a tension test is made. No changes in the tensile strength limits from those given in the first paper for high brass sheet and clock brass sheet have been found necessary.

In addition to the revision of the Rockwell limits for the two brass alloys mentioned above, four other brass alloys will be covered in this paper. One of these contains less lead than clock brass and is designed for use where a material combining moderate drawing and cutting properties is desired. Another has a nominal composition of 72 per cent of copper and 28 per cent of zinc. Grain size requirements are given for two others mainly used for drawing purposes. One of these consists of nominally 85 per cent of copper and 15 per cent of zinc and the other of 75 per cent of copper and 25 per cent of zinc. This paper also covers two nickel-silver alloys and two phosphor-bronze alloys.

The Rockwell hardness - tensile strength limits given for all of these additional materials may be considered tentative in view of the limited experience had with commercial shipments of materials purchased in accordance with these limits up to the present time. The requirements for high, clock and alloy E brass are considered final. Alloy E brass has a lead content midway between high and clock brass.

METHOD OF HANDLING WORK

As outlined in the previous paper, this investigation has been a cooperative one.

The round-robin tests on the Rockwell hardness tester, standard test blocks and calibration of the machines were made by the American Brass Co. (Waterbury, Buffalo and Kenosha Branches), Bell Telephone Laboratories, and the Western Electric Co. The tension tests were also made by these laboratories. All of the rolling series were manufactured by the American Brass Co. Experience data on shipments of non-ferrous metals were obtained jointly by the American Brass Co. and the Western Electric Co.

In addition to the authors the following took an important part in the investigational work. Messrs. H. N. Van Deusen and C. H. Greenall of Bell Telephone Laboratories, Messrs. W. H. Bassett, R. M. Tree, Alden Merrill and G. S. Mallett of the American Brass Co., and Messrs. M. D. Helfrick and G. R. Brown of the Western Electric Co. Messrs. N. E. Newton and W. H. Eastlake of the Northern Electric Co., Montreal, participated in the conferences which were held as the work progressed.

SCOPE OF INVESTIGATION

This paper is concerned with two projects: first, the further development of satisfactory commercial forms of test; and second, the determination of limits. The methods of test investigated and developed will be reviewed, and will be followed by presentation of the data upon which are based the test limits.

METHODS OF TEST

Tension Test:

The tension tests were carried on in the same manner as described in Appendix II to the previous paper. This method has been in use for over four years and is entirely satisfactory to the various laboratories participating in this investigation.

In order to evaluate the test results, it is necessary to establish control limits in order to determine whether any variations in the data are significant. By significant variations are meant variations that can be assigned to definite sources, such as measuring errors, defects in the metal tested, testing errors, etc. For the case under consideration, a method of analysis was employed which allows for small sample numbers. The method used is described elsewhere.[1]

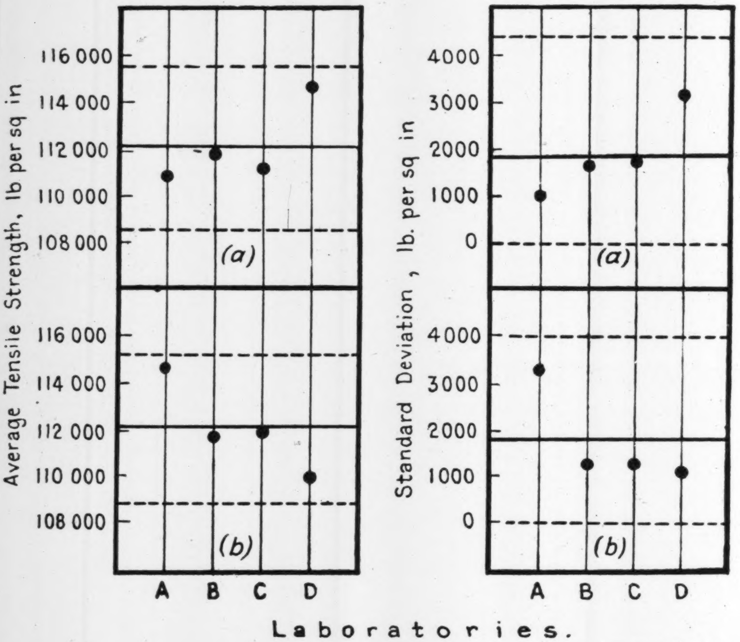

FIG. 1.—Statistical Analysis of Round-Robin Tension Tests on Alloy B Nickel Silver.

(*a*) Points are average of all specimens milled at laboratories indicated, and tested by the four laboratories.

(*b*) Points are average of all specimens tested at laboratories indicated, and milled at the several laboratories.

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[1] See Modified Criterion No. 1 in Appendix I to the Report of Sub-Committee XV on Die-Cast Metals and Alloys appended to the report of Committee B-2 on Non-Ferrous Metals and Alloys, *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part. I, p. 200 (1929).

Figure 1 shows an engineering analysis of the tensile strength results for alloy B nickel silver plotted in two ways.[1] The data in Fig. 1 (*a*) were obtained by computing, for each of the laboratories, the average tensile strength and the standard deviation[1] of all the specimens milled by that laboratory and tested by each of the four laboratories *A*, *B*, *C* and *D*. The data in Fig. 1 (*b*) were obtained by computing, for each of the laboratories, the average tensile strength and the standard deviation of the specimens milled in the four laboratories and tested in each of the respective laboratories. In each instance the specimens tested or milled by an individual laboratory are grouped together. The dotted lines on the diagrams represent the control limits within which the data should fall without leaving anything to chance, which means that points falling without these control limits indicate variations due to assignable causes, such as errors in measurements, defects in the material, etc. These diagrams show, therefore, that the tensile strength results between laboratories do not reveal significant or assignable difficulties other than those which could be attributed to chance. In other words, the analysis gave no indication of the presence of assignable variations between the testing laboratories.

These specimens were prepared by cross milling the gage length with a milling cutter shaped to conform to the final shape of the specimen desired. This method, which has been described elsewhere,[2] results in the saving of time and produces specimens of a uniform character.

Rockwell Hardness Tests:

The Rockwell hardness test is carried on in the same manner as given in Appendix I to the previous paper. All of the Rockwell hardness tests were made on the two grip ends of the tension test specimens. This was done in order to have Rockwell hardness tests and tension tests made on as nearly identical material as possible. Commercial experience with the Rockwell machine in the routine testing of sheet non-ferrous metals has indicated the importance of the following precautions.

Ball Penetrator.—Slight variations in the size and sphericity of the ball penetrators are to be expected and must be guarded against. The results of several hundred measurements made at various points on the "B" scale show that a deviation of less than 0.00002 in. in the diameter of the $\frac{1}{16}$-in ball has no effect on the hardness readings. Balls showing a greater variation give noticeable error in hardness readings. Penetrator balls have been held within this limit.

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[1] The methods used are those described by W. A. Shewhart in a paper on "Quality Control," *The Bell System Technical Journal*, Vol. VI, No. 4, p. 722, October, 1927.

[2] R. L. Templin, "Methods for Determining the Tensile Properties of Thin Sheet Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 235 (1927).

Anvil Surface.—It is desirable that the penetrator be perpendicular to the testing surface of the anvil. It is assumed that the pentrator is operating perpendicular to the seating surface and in line with the capstan head. Any lack of perpendicularity would then be due to lack of parallelism of the seating and testing surface of the anvil. This is checked by a fixture designed to hold the anvil so that the specimen supporting surface might be checked. A hardened flat, 2 in. square and $\frac{7}{8}$ in. thick, having a $\frac{13}{16}$-in. hole through its center, was lapped so that its two surfaces were flat and parallel to within 0.00005 in. By placing the fixture holding the anvil on the table of an Optimeter and sliding the supporting surface of the anvil under a ball-pointed feeler gage, errors in flatness and parallelism may be measured to 0.00005 in. Anvils whose surfaces were found to be plane to within 0.0001 in. were considered satisfactory.

Machine Errors.—Observations on four Rockwell machines at the Western Electric Co., one of which handles approximately 10,000 tests per month and all of which were overhauled and calibrated for their entire range once a week, showed that changes in calibration occuring in this interval were less than one hardness number. Before beginning a series of readings the correction to be applied was determined by taking readings on a standard block. The average change on all three scales was about one hardness number. Variations from calibration were rarely greater than 1.5 numbers unless the penetrator was damaged or the instrument was out of adjustment due to misuse.

The effectiveness with which standard test blocks may be used in bringing Rockwell testers into agreement is shown by the following calibration:

	APPROXIMATE ROCKWELL HARDNESS, "B" SCALE, $\frac{1}{16}$-IN. BALL, 100-KG. LOAD (RED FIGURES)		
	25	60	80
Before overhauling and checking the entire scale, average difference in hardness readings between three testers	6.0	3.7	3.4
After overhauling and checking the entire scale, average difference in hardness readings between three testers	0.9	1.2	0.9

Calibration of Test Blocks.—At the time of the publication of the first paper it was believed on the basis of the data then available that a Rockwell hardness machine would hold its adjustment over a long period if carefully maintained and could serve as a standard machine to be used for the calibration of test blocks. More complete experience has shown, however, that wear of the operating parts and slight variations in friction caused the "standard machine" to vary slightly in its readings. Naturally, the calibration of standard test blocks under these conditions is unsatisfactory because it might result in the establishment of several sets of standard blocks, leading to confusion in the application of the Rockwell hardness test limits to commercial material.

A new method of calibrating the Rockwell hardness tester was devised. This method consisted in preparing a set of test blocks covering the range of hardness of the high brass and clock brass rolling series. These blocks were checked by the various cooperating laboratories as well as the manufacturers of the Rockwell hardness tester, namely, the Wilson-Maeulen Co. These blocks were considered the basic standards for the Rockwell tests. Sub-standard blocks were calibrated in comparison with these standard blocks and placed in use by the various cooperating laboratories. The high brass rolling series and the clock brass rolling series were then retested for Rockwell hardness using the new calibration. It was found that the hardness readings differed from one to two points from the values for several tempers given in the previous paper. At the same time experience with commercial shipments of material showed that a shift was necessary in the Rockwell hardness - tensile strength relationship. This verified our conclusions with regard to the adoption of the new method of calibration.

After the establishment of the standard brass test blocks it was seen that blocks would also be required to cover the nickel-silver and the phosphor-bronze alloys. To prepare new series of blocks especially for these materials would have resulted in a wide variety of blocks and would have been found burdensome to the laboratories. Consequently, a single series of test blocks covering the range of hardness testing was selected. This series of test blocks started with the original blocks to which were added a few additional blocks made necessary in order to cover the range of the nickel-silver and phosphor-bronze alloys. The data upon which the calibration of these blocks was based have been submitted to the Section on Indentation Hardness of the Society's Committee E-1 on Methods of Testing and with the cooperation of the U. S. Bureau of Standards it is believed that this information will be applied along with other information available for the standardization of national standard test blocks.

Hardness of Thin Sheet.—The Rockwell hardness test has certain limitations in its application to the testing of thin sheet stock. Material less than 0.020 in. thick gives hardness readings different from thicker sheet of the same temper. Referring to the curves shown hereinafter of Rockwell hardness plotted against tensile strength for materials thinner than No. 24 B. & S. gage the points fall below the curve. This apparently is due to lack of support of the metal about the penetrator and consequently a low reading is given. For still thinner material the penetrator passes nearly through the metal and the hardness reading recorded is inaccurate due to the effect of the supporting anvil in addition to lack of support of the metal.

TABLE I.—ROCKWELL HARDNESS TESTS ON ALLOY G BRASS SHEET SHOWING EFFECT OF USING UNPOLISHED AND POLISHED ANVIL.

Sample		Tensile Strength, lb. per sq. in.	Rockwell Hardness, Unpolished Anvil, "B" Scale, $\frac{1}{16}$-in. Ball, 100-kg. Load (Red Figures)				Rockwell Hardness, Polished Anvil, "B" Scale, $\frac{1}{16}$-in. Ball, 100-kg. Load (Red Figures)			
B. & S. Gage	Temper, B. & S. Numbers Hard		Average of 15 Readings	Maximum	Minimum	Range	Average of 15 Readings	Maximum	Minimum	Range
No. 20	2	64 000	73.3	74.0	72.8	1.2	72.9	73.6	71.8	1.8
	4	77 800	83.5	83.8	82.9	0.9	83.5	83.7	83.0	0.7
	6	85 400	87.0	87.6	86.2	1.4	86.7	86.7	86.0	0.7
	8	93 800	90.3	90.0	90.0	0.8	90.7	90.8	90.3	0.5
	10	99 000	92.1	92.7	91.3	1.4	92.7	92.8	92.6	0.2
No. 22	2	65 100	74.2	75.0	73.6	1.4	72.6	73.5	71.3	2.2
	4	79 100	83.4	83.8	83.2	0.5	83.8	84.2	83.0	1 2
	6	88 200	88.3	88.9	88.0	0.8	88.0	88.3	87.8	0.5
	8	99 300	90.0	90.2	89.7	0.5	90.1	90.2	89.8	0.4
	10	98 200	91.8	92.2	91.7	0.5	92.2	92.4	91.8	0.6
No. 24	2	61 000	67.6	68.8	67.1	1.7	63.9	65.3	61.6	3.7
	4	81 600	84.3	84.7	83.7	1.0	82.7	83.0	82.2	0.8
	6	91 400	89.0	89.7	88.7	1.0	87.8	88.0	87.6	0.4
	8	96 600	91.0	91.2	90.7	0.5	89.7	89.9	89.3	0.6
	10	97 800	91.9	92.7	91.2	1.5	91.2	91.5	90.9	0.6
No. 26	2	58 100	64.4	65.9	63.0	2.9	58.4	59.4	56.8	2.6
	4	71 800	77.3	77.8	76.9	0.9	75.1	75.5	74.4	1.1
	6	88 600	87.4	88.0	86.7	1.3	86.2	86.5	85.8	0.7
	8	95 600	90.2	90.7	89.9	0.8	89.5	89.9	89.2	0.7
	10	99 100	91.2	91.7	90.8	0.9	91.0	91.3	90.6	0.7
No. 28	2	61 100	65.4	66.9	64.2	2.7	55.7	57.6	52.3	5.3
	4	71 400	77.0	77.9	76.0	1.7	72.1	73.3	71.1	2.2
	6	83 600	83.2	83.8	82.1	1.7	81.2	81.5	80.7	0.8
	8	94 700	89.6	90.3	89.1	1.2	88.2	88.5	88.0	0.5
	10	100 100	91.5	92.0	90.5	1.5	90.4	90.8	90.1	0.7

Cleaning Anvil and Specimen.—Inasmuch as the Rockwell hardness tester measures hardness in terms of penetration, any movement of the penetrator affects the hardness reading. In other words, if the metal has a roughened surface or if the anvil is not polished smooth the metal will flow under the high unit pressure involved and this will cause the Rockwell hardness reading to be lower than its true value. It is considered necessary therefore that the anvil should be polished flat and the material tested should be reasonably free from surface imperfections and oxide film. Table I shows the effect on the hardness readings of polishing the anvil.

In addition to the need of polishing the anvils, if close agreement is to be had, some refinement is also needed in the use of the standard test blocks. It is difficult to obtain test blocks that will not show a variation in hardness. It has been customary, therefore, in calibrating test blocks to take five readings, one in each corner of the test block and one in the center. These readings are made on a machine which has been calibrated with the standard test blocks both before and after calibration of the secondary blocks. Experience in calibrating the Rockwell hardness machine with standard test blocks has emphasized the need for keeping these blocks clean and free from oxide, grease or other accumulated matter. This applies with equal force

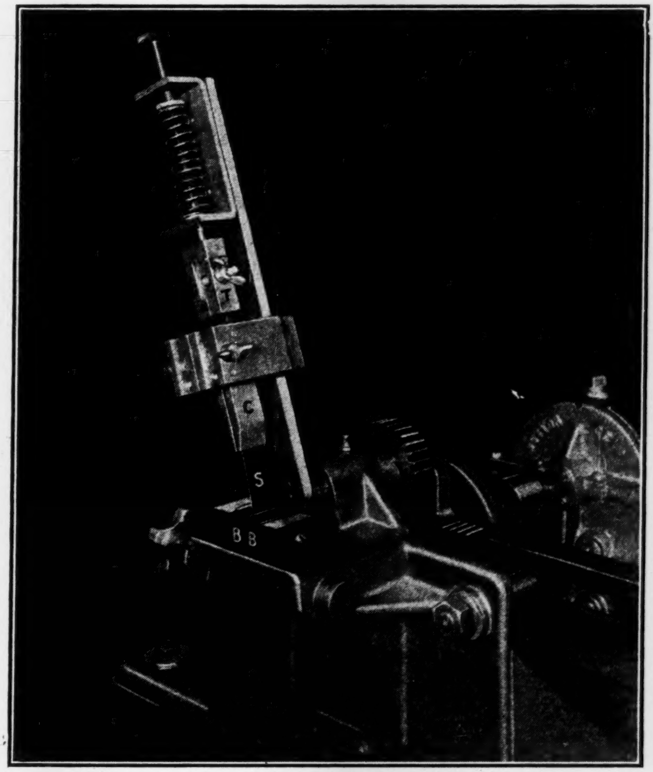

FIG. 2.—Experimental Bend Testing Machine.

T, tensioning grip.
C, guide clamp.
S, specimen.
B, radius block.

to the test specimens. Various methods of cleaning the test blocks have been tried. A successful and convenient method is to rub the surfaces of the test block by hand with chiffon velvet or other lap material dipped in tripoli and water. Test blocks cleaned in this way do not show a greater variation in readings than that of the original test for uniformity.

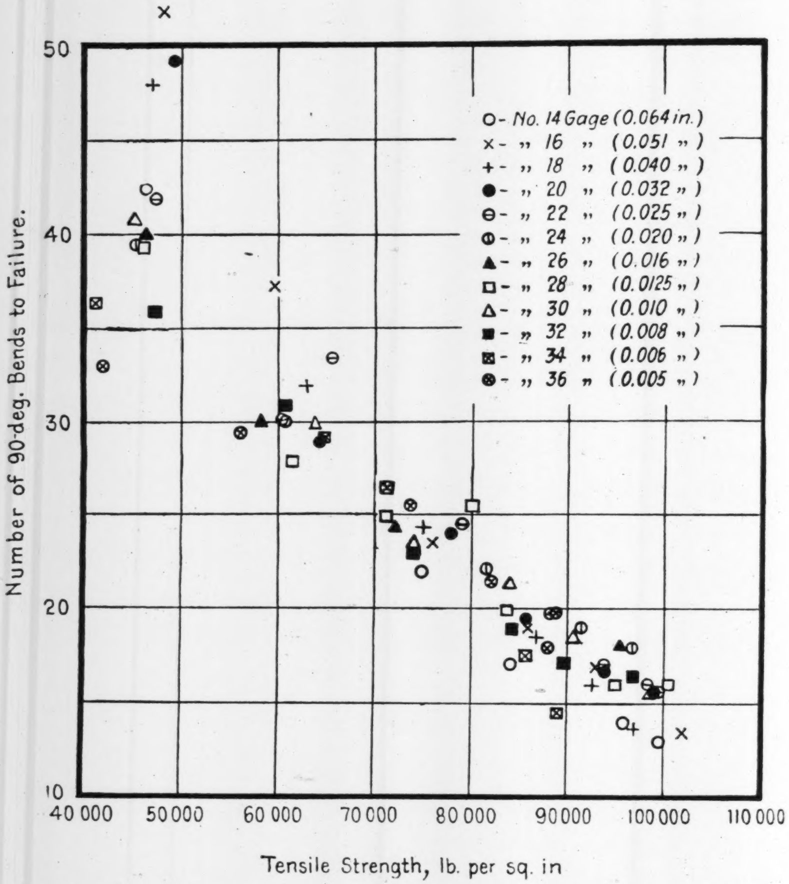

FIG. 3.—Relation of Number of 90-deg. Bends to Failure to Tensile Strength of Alloy G Brass. Ratio of Mandrel Radius to Thickness, 8 to 1.

A variation in hardness has to be permitted in these five readings and on the basis of experience with a large number of blocks it is considered that test blocks should have no greater variation than as follows: When tested using the "B" scale, $\frac{1}{16}$-in. ball, 100-kg. load (red figures), for blocks under 40, 3 points variation is allowed; for blocks from 40 to 60, $2\frac{1}{2}$ points; and for blocks over 60, $1\frac{1}{2}$ points. Acceptance or rejection of blocks is based on these limits, except where a minus reading results and in this case the 60-kg. load is used. Blocks of this uniformity are calibrated for use with the 60, 100 and 150-kg. loads.

Rockwell Scales Employed.—It has been found that the Rockwell "B" scale employing the 100-kg. load, the $\frac{1}{16}$-in. diameter ball and reading the red figures, is satisfactory for rolled brass sheet; but in the case of the nickel-silver and phosphor-bronze alloys it was found that the Rockwell hardness - tensile strength curve became asymptotic, showing very little change in hardness for a large increase in tensile strength in the harder tempers. A load of 150 kg. was substituted for the 100-kg. load and resulted in an improvement since by using the larger load, a greater depth of penetration was obtained and consequently better sensitivity in the higher tempers. The curve, shown by Fig. 10, gives the Rockwell hardness - tensile strength relationship employing the 100 and 150-kg. loads on the Rockwell tester. The 150-kg. load has been adopted for nickel-silver and phosphor-bronze alloys and the Rockwell hardnesses reported hereinafter for these materials are the values obtained using a 150-kg. load, a $\frac{1}{16}$-in. diameter ball and reading on the "B" scale or red figures. The 60-kg. load is used for testing annealed material. (See Table II.)

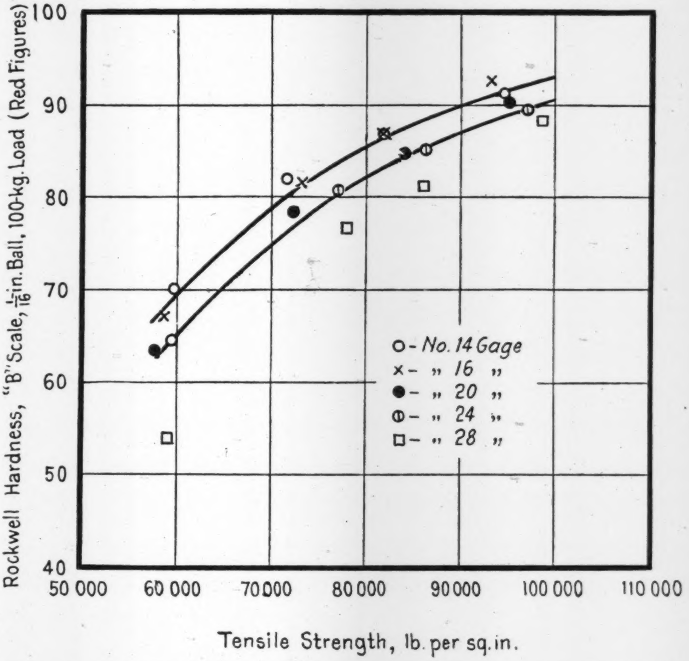

FIG. 4.—Relation of High Brass Rolling Series. Rockwell Hardness to Tensile Strength Retested, Using Standard Rockwell Blocks.

Fatigue Tests:

Each of the sheet metals covered by this paper have had their endurance limits determined in the annealed condition and as rolled four and ten numbers hard. The results of these tests are covered by another paper.[1]

Bend Test Machine:

Continuing the work on the Amsler bend test machine mentioned in the previous paper, an improvement on this machine has been developed employing a motor drive and accurately prepared and aligned jaws. Figure 2 shows a model of this machine built for experimental purposes. A strip of metal *S*, ½ in. wide, is clamped between a pair of radius blocks, *B*, selected at random. The upper portion of the specimen is held by a tensioning grip, *T*, and guided by two clamps, *C*. The specimen is bent back and forth over the pair of radius blocks and each bend of 90 deg. and return is recorded as one bend by a counter. Using another strip of the same metal but a pair of radius blocks of different radii another result is obtained. If now the number of bends is plotted on log paper against the ratio of the thickness of the metal to the radius of the blocks, a straight line results. It is possible therefore to select any standard ratio of block radius to thickness of metal and by two tests employing two different pairs of radius blocks to determine the number of bends for this predetermined ratio by interpolation. The ratio finally selected as most representative was 8 to 1. Figure 3 gives the bend test results for alloy G brass.

The bend test reveals the ductility and toughness of the metal under test. It should show a close correlation to the forming and drawing properties of sheet metal. Further studies will be required to work out such relationships.

Grain Count:

A series of photomicrographs of annealed brass has been adopted covering the grain sizes 0.010 to 0.200 mm. diameter of average grain.[2] Inasmuch as the alpha phase grain structures of the nickel-silver and phosphor-bronze alloys are similar to those of brass, one set of standards may be employed for these metals. They were carefully counted by the Jeffries method as described in the note to Section 9 of the Society's Standard Rules Governing the Preparation of Micrographs of Metals and Alloys (E 2 - 27)[1] and are taken at a magnification of 75 diameters. Grain sizes of specimens under examination are determined by comparing these standard photomicrographs.

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[1] J. R. Townsend and C. H. Greenall, "Fatigue Studies of Non-Ferrous Sheet Metals," see p. 353.

[2] See Report of Committee E-4 on Metallography, *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part I, p. 505 (1929).

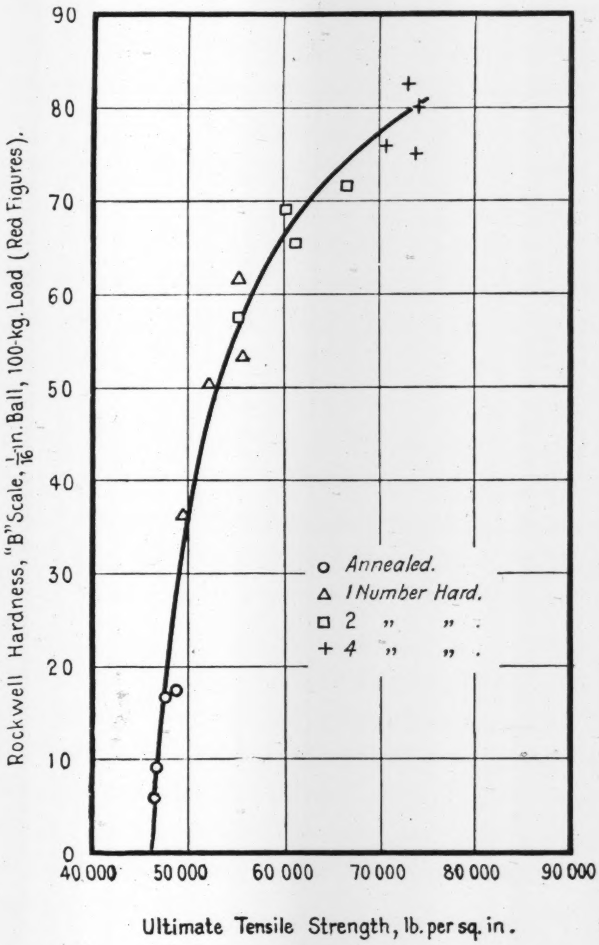

FIG. 5.—Relation of Rockwell Hardness to Tensile Strength of Alloy E Brass.

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[1] 1927 Book of A.S.T.M. Standards, Part I, p. 778.

TABLE II.—AVERAGE GRAIN DIAMETER[a] FOR ANNEALED BRASS AND NICKEL SILVER.

Material	Anneal	Average Tensile Strength, lb. per sq. in.	Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in. Ball, 60-kg. Load, (Red Figures) Maximum	Average Grain Diameter, mm.	
				Minimum	Maximum
High Brass	Light	50 000	80	0.010	0.035
	Drawing	47 000	65	0.035	0.090
	Soft Drawing	43 000	55	0.090	0.200
Alloy C Brass	Light	42 000	76	0.010	0.030
	Drawing	40 000	61	0.030	0.070
	Soft Drawing	39 000	55	0.070	0.150
Alloy D Brass	Light	53 000	82	0.010	0.030
	Drawing	49 000	72	0.030	0.075
	Soft Drawing	45 000	59	0.075	0.200
Alloy E Brass	Light	50 000	80	0.010	0.035
	Drawing	46 000	67	0.035	0.080
	Soft Drawing	44 000	57	0.080	0.160
Alloy A Nickel Silver	Light	55 000	87	0.010	0.025
	Drawing	54 000	80	0.025	0.050
	Soft Drawing	52 500	77	0.040	0.100

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[a] The method for determining the grain size of the material is the Jeffries method as described in the note under Section 9 of the Society's Standard Rules Governing the Preparation of Micrographs of Metals and Alloys (E 2 - 27), see 1927 Book of A.S.T.M. Standards, Part I, p. 778.

TABLE III.—REVISED ROCKWELL HARDNESS AND TENSILE STRENGTH LIMITS FOR HIGH, CLOCK AND ALLOY E BRASS SHEET.

Thickness	Temper, B. & S. Numbers Hard	Percentage Reduction by Rolling	Tensile Strength,[a] lb. per sq. in.						Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in. Ball, 100-kg. Load (Red Figures), for High, Clock and Alloy E Brass Sheet	
			High Brass		Clock Brass		Alloy E Brass			
			Minimum	Maximum	Minimum	Maximum	Minimum	Maximum	Minimum	Maximum
0.040 in. and over	Quarter hard..... 1	10.9	46 000	56 000	47 000	57 000	47 000	57 000	30	60
0.020 to 0.040 in.		10.9	46 000	56 000	47 000	57 000	47 000	57 000	30	56
0.040 in. and over	Half hard..... 2	20.7	53 500	63 500	54 500	64 500	54 000	64 000	50	73
0.020 to 0.040 in.		20.7	53 500	63 500	54 500	64 500	54 000	64 000	49	71
0.040 in. and over	Three-quarters hard.. 3	29.5	61 000	71 000	.....	.....	.....	.....	70	80
0.020 to 0.040 in.		29.5	61 000	71 000	.....	.....	.....	.....	67	77
0.040 in. and over	Hard..... 4	37.1	68 000	78 000	68 000	78 000	68 000	78 000	78	85
0.020 to 0.040 in.		37.1	68 000	78 000	68 000	78 000	68 000	78 000	75	83
0.040 in. and over	Extra hard..... 6	50.0	79 000	88 500	79 000	88 500	.....	.....	85	89
0.020 to 0.040 in.		50.0	79 000	88 500	79 000	88 500	.....	.....	83	87
0.040 in. and over	Spring... 8	60.5	86 000	95 000	85 000	94 500	.....	.....	88	92
0.020 to 0.040 in.		60.5	86 000	95 000	85 000	94 500	.....	.....	85	89
0.040 in. and over	Extra spring....10	68.7	89 500	98 500	88 000	97 500	.....	.....	89	93
0.020 to 0.040 in.		68.7	89 500	98 500	88 000	97 500	.....	.....	86	90

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[a] Tensile strength values for high and clock brass sheet are the same as in the previous paper by H. N. Van Deusen, L. I. Shaw and C. H. Davis, "Physical Properties and Methods of Test for Sheet Brass," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 173 (1927).

For the nickel-silver alloys, a magnification of 150 is employed because of the smaller grain size of this material compared with brass.

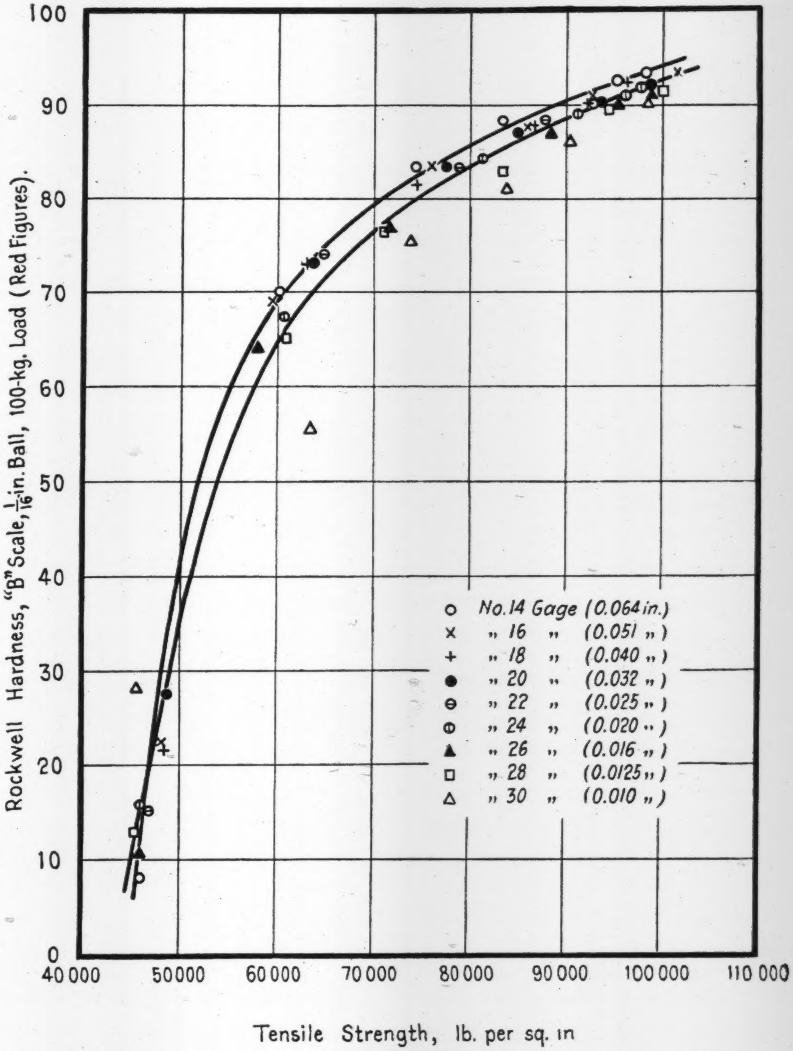

FIG. 6.—Relation of Rockwell Hardness to Tensile Strength of Alloy G Brass Rolling Series.

The limits of diameter of average grain of the alloys used for drawing purposes are given in Table II, namely for high brass sheet, alloys C, D, and E brass, and alloy A nickel silver. (See Table IV for chemical composition.)

DETERMINATION OF LIMITS

Revision of Rockwell Hardness Limits for High and Clock Brass:

As mentioned above, Rockwell hardness limits given in the previous paper have been revised to agree with the more recently established Rockwell standards which have been verified by more experience data collected over several years and representing material furnished under the limits by a number of suppliers. Table III shows the new Rockwell hardness limits. Figure 4 shows Rockwell hardness plotted against tensile strength for the high brass rolling series retested using standard blocks. There have been no changes in tensile strength limits. Figure 16 shows Rockwell hardness plotted against tensile strength, for high and clock brass the individual points rep-

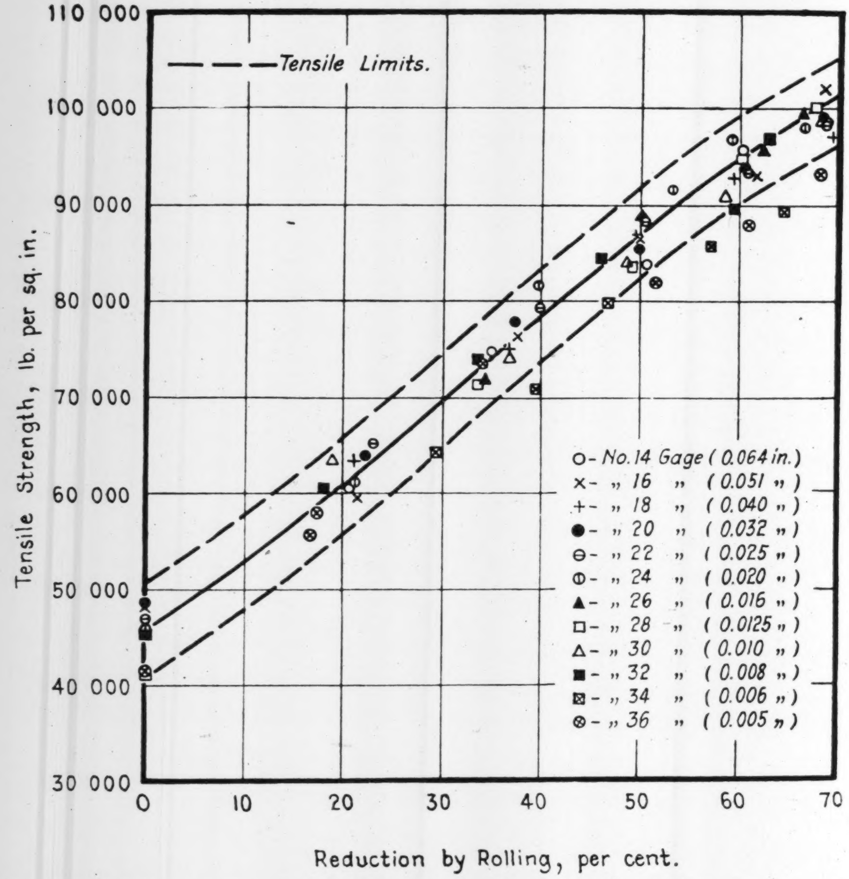

FIG. 7.—Relation of Tensile Strength to Percentage of Reduction by Rolling of Alloy G Brass Rolling Series.

resenting determinations of Rockwell hardness and tensile strength on commercial shipments of material. The lines drawn show the grouping of the data according to thickness.

TABLE IV.—CHEMICAL COMPOSITION LIMITS FOR NON-FERROUS METAL SHEET.

		High Sheet Brass	Clock Brass	Alloy C Brass	Alloy D Brass	Alloy E Brass	Alloy G Brass	Alloy A Nickel Silver	Alloy B Nickel Silver	Alloy A Phosphor Bronze	Alloy C Phosphor Bronze
Copper, per cent...	minimum	64.50	61.00	83.00	73.00	64.00	70.50	70.50	53.50	94.40	91.00
	maximum	67.50	64.00	86.00	76.00	67.00	73.50	73.50	56.50	....	....
Lead, per cent...	minimum	0.00	1.25	....	....	0.80	....	....	....	....	....
	maximum	0.30	2.00	0.15	0.25	1.10	0.10	....	0.10	0.05	0.02
Iron, per cent...	minimum	0.00	0.00	....	....	....	....	....	....	....	....
	maximum	0.05	0.06	0.05	0.05	0.08	0.05	0.35	0.35	0.10	0.10
Zinc, per cent...	minimum	Rem'der	Rem'der	Rem'der	Rem'der	Rem'der	Rem'der	8.50	25.50	....	....
	maximum	Rem'der	Rem'der	Rem'der	Rem'der	Rem'der	Rem'der	11.50	28.50	0.30	0.20
Nickel, per cent...	minimum	....	....	....	....	....	....	16.50	16.50	....	....
	maximum	....	....	....	....	....	....	19.50	19.50	....	....
Manganese, per cent...	minimum	....	....	....	....	....	....	....	....	....	....
	maximum	....	....	....	....	....	....	....	0.50	....	....
Tin, per cent...	minimum	....	....	....	....	....	....	....	....	3.80	7.50
	maximum	....	....	....	....	....	....	....	....	4.80	8.50
Phosphorus, per cent...	minimum	....	....	....	....	....	....	....	....	0.05	0.05
	maximum	....	....	....	....	....	....	....	....	0.35	0.25
Antimony, per cent...	minimum	....	....	....	....	....	....	....	....	....	....
	maximum	....	....	....	....	....	....	....	....	0.01	0.01
Other Impurities, per cent...	minimum	....	....	....	....	....	....	....	....	....	....
	maximum	0.10	0.10	....	....	0.10	0.10	....	....	trace	trace

TABLE V.—CHEMICAL ANALYSES OF ROLLING SERIES.

	Alloy E Brass	Alloy G Brass	Alloy A Nickel Silver	Alloy B Nickel Silver	Alloy A Phosphor Bronze	Alloy C Phosphor Bronze
Copper, per cent	66.02	71.73	71.31	55.23	95.35	91.84
Lead, per cent	1.08	0.02	....	0.005	0.01	0.02
Iron, per cent	0.03	0.03	0.12	0.06	0.05	0.03
Zinc, per cent	32.87	28.21	10.74	26.27	0.00	0.00
Nickel, per cent	....	0.01	17.62	18.38	0.00	0.00
Manganese, per cent	....	....	0.12	0.11	....	....
Tin, per cent	....	0.00	....	....	4.48	8.08
Phosphorus, per cent	....	....	....	....	0.08	0.03
Graphite, per cent	....	....	0.00	0.00	....	....
Combined Carbon, per cent	....	....	0.013	0.018	....	....

Alloys C and D Brass Sheet:

The chemical composition limits for alloys C and D brass sheet are given in Table IV. The grain size limits given in Table II are based on the range of commercial annealing practice. Inasmuch as these alloys are employed only for drawing purposes, no tensile strength limits are necessary. Maximum Rockwell hardness limits are given to exclude cold-worked metal.

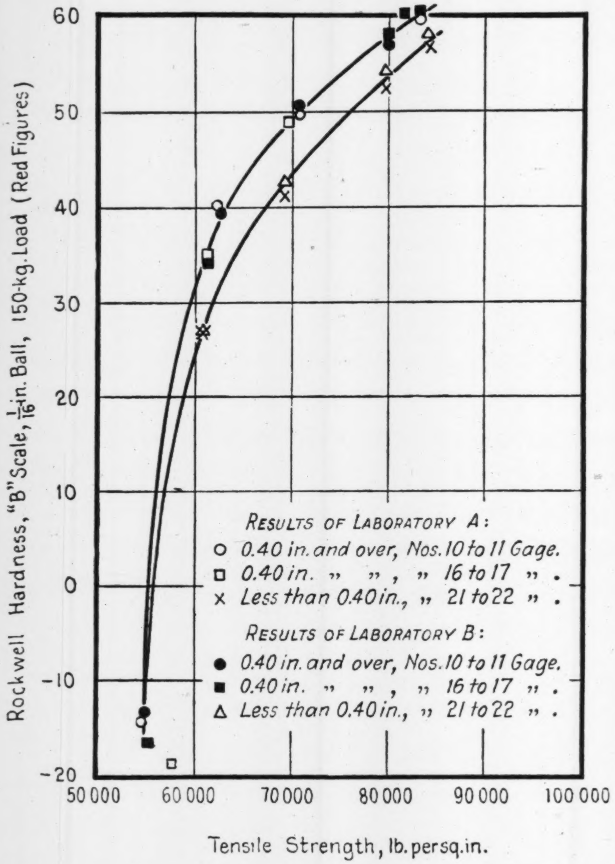

FIG. 8.—Relation of Rockwell Hardness to Tensile Strength of Alloy A Nickel Silver Rolling Series.

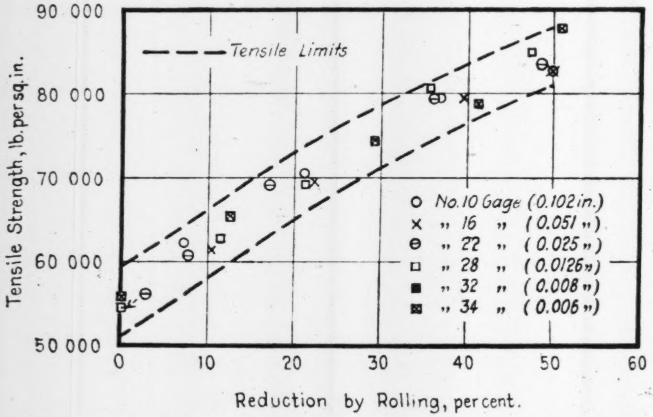

FIG. 9.—Relation of Tensile Strength to Percentage of Reduction of Alloy A Nickel-Silver Rolling Series.

Alloy E Brass Sheet:

Four rolling series were made from one bar of metal having the analysis shown in Table V. These series begin at B. & S. gage Nos. 10, 14, 18 and 22 and were rolled 1, 2 and 4 B. & S. gage numbers from commercial anneals in the 500 to 650° C. range. Figure 5 shows Rockwell hardness plotted against tensile strength for alloy E brass.

In the previous paper it was shown that different Rockwell hardness values were required for material 0.040 in. and thicker, and materials less than 0.040 in. thick. This change in limits is dependent upon thickness and is not sharply defined, but it has been found sufficiently accurate for all practical purposes to consider that the division occurs at 0.040 in. In the case of the alloy E brass rolling series results shown by Fig. 5, only one curve was drawn since not sufficient data were available to show this division. The curve practically agrees with the curves given in the previous paper[1] for the high brass rolling series, and also with the data shown on Fig. 16 for commercial shipments of high and clock brass.

TABLE VI.—PHYSICAL PROPERTIES OF ALLOY E BRASS SHEET.

B. & S. Gage	Temper, B. & S. Numbers Hard	Thickness, in.	Percentage Reduction by Rolling		Proportional Limit, lb. per sq. in.	Tensile Strength, lb. per sq. in.	Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in. Ball, 100-kg. Load (Red Figures)	Modulus of Elasticity, lb. per sq. in.	Elongation in 2 in., per cent
			At Edges	At Center					
No. 10......	0	0.1057	0	0	......	48 330	17.4	....	65.8
No. 11......	1	0.0903	14.2	15.2	20 050	55 130	61.6	13 740 000	41.0
No. 12......	2	0.0817	22.0	23.4	30 417	60 630	71.8	14 600 000	30.7
No. 14......	4	0.0657	37.2	38.5	27 230	72 930	82.6	16 540 000	10.8
No. 14......	0	0.0656	0	0	......	46 230	9.1	....	64.3
No. 15......	1	0.0568	10.6	10.6	18 525	52 000	50.3	16 460 000	46.5
No. 16......	2	0.0508	23.1	22.8	25 870	60 200	69.2	16 760 000	28.5
No. 18......	4	0.0402	38.5	38.5	29 670	74 066	80.1	14 480 000	8.0
No. 18......	0	0.0402	0	0	......	46 130	5.8	....	67.3
No. 19......	1	0.0371	9.2	9.3	18 870	49 600	36.3	....	52.3
No. 20......	2	0.0338	17.4	17.2	20 885	55 230	57.6	15 380 000	37.8
No. 22......	4	0.0260	36.7	36.3	23 385	70 530	76.0	17 000 000	11.2
No. 22......	0	0.0256	0	0	......	47 500	16.8	....	59.0
No. 23......	1	0.0225	15.8	14.1	25 410	55 500	53.2	12 340 000	38.7
No. 24......	2	0.0205	23.2	21.8	36 460	61 010	65.7	12 300 000	29.0
No. 26......	4	0.0169	36.7	35.1	34 090	74 050	75.2	....	7.5

NOTE.—Tensile strength and percentage elongation are an average of 3 specimens in each case; Rockwell hardness values are an average of 15 determinations of 5 readings on each of 3 tension test specimens.

The composition of alloy E brass is midway between that of high and clock brass, and consequently it was to be expected that the Rockwell hardness limits for this material would be the same as for the foregoing. The tension test is more sensitive to such changes in chemical composition and since it is the test upon which acceptance or rejection of material is based, separate tensile strength limits are given. The separate tensile strength and the combined Rockwell hardness limits for these alloys are given in Table III. The physical properties of alloy E brass are given in Table VI.

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[1] Van Deusen, Shaw and Davis, *loc. cit.*

TABLE VII.—PHYSICAL PROPERTIES OF ALLOY G BRASS SHEET.

B. & S. Gage	Temper, B. & S. Numbers Hard	Thickness, in.	Actual Percentage Reduction by Rolling	Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in. Ball, 100-kg. Load (Red Figures)	Tensile Strength, lb. per sq. in.	Elongation in 2 in., per cent
No. 14	0	0.0655	0	8.5	46 200	68
	2	0.0655	20.6	70.1	60 500	32.5
	4	0.0660	35.0	83.4	74 600	11
	6	0.0658	50.6	88.2	83 800	7.5
	8	0.0655	60.4	92.6	95 600	6.5
	10	0.0655	68.8	93.4	98 600	6.5
No. 16	0	0.0520	0	22.7	48 300	60
	2	0.0522	21.4	69.0	59 700	33
	4	0.0515	37.6	83.7	76 200	9
	6	0.0500	49.9	87.7	86 400	6
	8	0.0503	61.5	91.0	93 000	6.5
	10	0.0512	68.6	93.5	102 000	4.5
No. 18	0	0.0424	0	21.8	48 700	63
	2	0.0409	21.0	73.2	63 400	25.5
	4	0.0412	36.8	81.6	74 900	9.5
	6	0.0408	49.6	87.9	86 800	6
	8	0.0415	59.4	90.1	92 700	4
	10	0.0412	69.6	92.4	96 900	3
No. 20	0	0.0339	0	27.6	48 900	57
	2	0.0333	22.2	73.3	64 000	25
	4	0.0329	37.4	83.5	77 800	7.5
	6	0.0330	49.9	87.0	85 400	5
	8	0.0324	60.7	90.3	93 800	3
	10	0.0320	68.5	92.1	99 000	3
No. 22	0	0.0256	0	15.4	47 100	64
	2	0.0260	23.0	74.2	65 100	22.5
	4	0.0265	39.9	83.4	79 100	7.5
	6	0.0263	50.5	88.3	88 200	2.5
	8	0.0260	60.7	90.0	93 300	3
	10	0.0255	68.8	91.8	98 200	2.5
No. 24	0	0.0201	0	16.0	46 300	61
	2	0.0203	21.1	67.6	61 000	26
	4	0.0204	39.6	84.3	81 600	6
	6	0.0204	53.3	89.0	91 400	3.5
	8	0.0208	59.3	91.0	96 600	2
	10	0.0207	66.6	91.9	97 800	2
No. 26	0	0.0169	0	11.0	46 200	58.0
	2	0.0167	17.3	64.4	58 100	26.5
	4	0.0168	34.3	77.3	71 800	9
	6	0.0168	50.9	87.4	88 600	2
	8	0.0165	62.4	90.2	95 600	1.5
	10	0.0172	66.5	91.2	99 100	1.5
No. 28	0	0.0133	0	13.2	45 800	61.5
	2	0.0135	21.1	65.4	61 100	24
	4	0.0134	33.5	77.0	71 400	8
	6	0.0135	49.0	83.2	83 600	2
	8	0.0135	60.1	89.6	94 700	1.5
	10	0.0132	67.7	91.5	100 100	1
No. 30	0	0.0108	0	28.5	45 600	54
	2	0.0103	18.8	55.7	63 600	20
	4	0.0106	36.6	75.7	74 000	4
	6	0.0106	48.7	81.2	84 000	2
	8	0.0108	58.6	86.1	90 600	1.5
	10	0.0108	68.4	90.3	98 600	1
No. 32	0	0.0085	0	....	46 800	51.5
	2	0.0084	17.8	....	60 500	24.5
	4	0.0085	33.6	....	73 900	5
	6	0.0087	46.0	....	84 300	1.5
	8	0.0086	59.4	....	89 700	1
	10	0.0085	63.0	....	96 700	0.5
No. 34	0	0.0071	0	....	41 200	39
	2	0.0065	29.2	....	64 400	20.5
	4	0.0071	39.6	....	70 900	4.5
	6	0.0071	46.9	....	79 800	1.5
	8	0.00705	57.1	....	85 700	1
	10	0.0072	64.5	....	89 200	0.75
No. 36	0	0.0057	0	....	41 600	33
	2	0.0054	16.7	....	55 700	18
	4	0.0056	34.1	....	73 400	4
	6	0.0056	51.5	....	81 900	2
	8	0.0056	60.9	....	87 700	2
	10	0.0052	68.3	....	93 100	1

NOTE.—Tensile strength and percentage elongation are an average of 3 specimens in each case; Rockwell hardness values are an average of 15 determinations of 5 readings on each of 3 tension test specimens.

It will be noted that the Rockwell hardness - tensile strength limits for alloy E brass are given only for material 1, 2 and 4 numbers hard. This alloy is not normally used in higher tempers as it is especially adapted for use where both forming and machining operations are involved, so that higher leaded or harder materials would not be suitable.

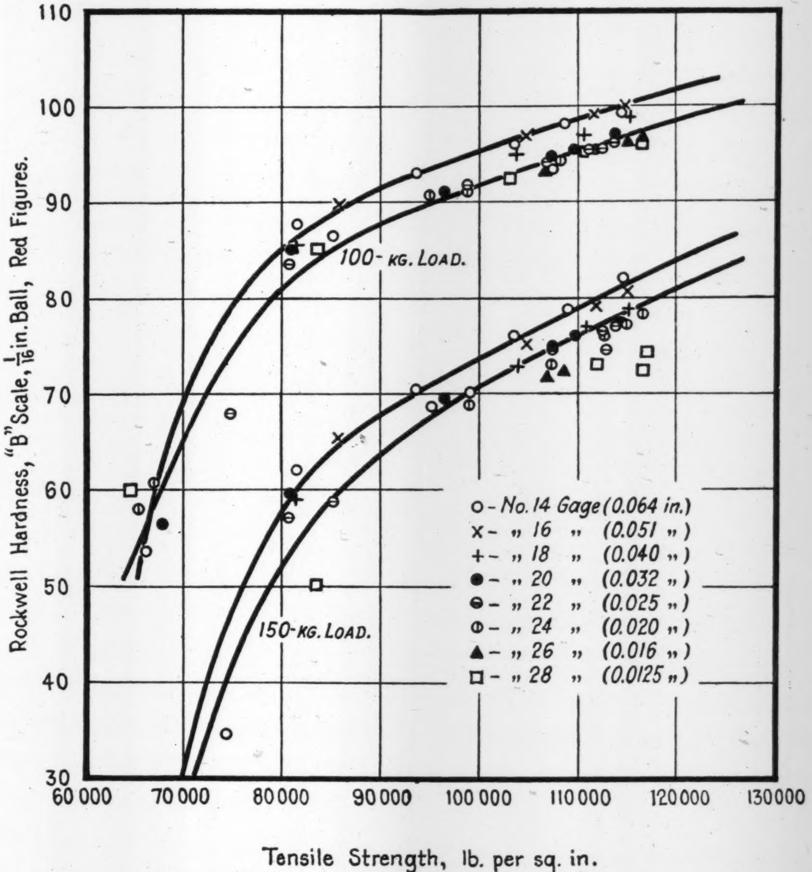

FIG. 10.—Relation of Rockwell Hardness to Tensile Strength of Alloy B Nickel-Silver Rolling Series Using 100 and 150-kg. Loads.

The chemical composition requirements for alloy E brass sheet are given in Table IV.

Alloy G Brass Sheet:

It has been pointed out in a previous paper by one of the authors[1] that this material, consisting nominally of 72 per cent of copper and

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[1] W. H. Bassett and C. H. Davis, "Physical Characteristics of Copper and Zinc Alloys," *Proceedings*, Inst. Metals Div., Am. Inst. Mining and Metallurgical Engrs. (1928).

TABLE VIII.—ROCKWELL HARDNESS AND TENSILE STRENGTH LIMITS FOR ALLOY G BRASS SHEET.

Thickness	Temper, B. & S. Numbers Hard	Percentage Reduction by Rolling	Tensile Strength, lb. per sq. in.		Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in. Ball, 100-kg. Load (Red Figures)	
			Minimum	Maximum	Minimum	Maximum
0.040 in. and over....	Quarter hard... 1	11.0	49 000	59 000	35	67
0.020 to 0.040 in.....		11.0	49 000	59 000	30	63
0.040 in. and over....	Half hard...... 2	20.7	56 500	66 500	63	76
0.020 to 0.040 in.....		20.7	56 500	66 500	57	73
0.040 in. and over....	Hard.......... 4	37.1	71 000	81 000	80	86
0.020 to 0.040 in.....		37.1	71 000	81 000	77	84
0.040 in. and over....	Extra hard.... 6	50.0	82 500	91 500	87	91
0.020 to 0.040 in.....		50.0	82 500	91 500	85	89
0.040 in. and over....	Spring........ 8	60.5	90 500	99 500	90	94
0.020 to 0.040 in.....		60.5	90 500	99 500	88	92
0.040 in. and over....	Extra spring...10	68.7	95 000	104 000	92	96
0.020 to 0.040 in.....		68.7	95 000	104 000	90	94

TABLE IX.—PHYSICAL PROPERTIES OF ALLOY A NICKEL-SILVER SHEET.

B. & S. Gage	Temper, B. & S. Numbers Hard	Thickness, in.	Actual Percentage Reduction by Rolling	Elongation in 2 in., per cent	Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in Ball (Red Figures)		Tensile Strength, lb. per sq. in.
					100-kg. Load	150-kg. Load	
No. 10..........	0	0.1034	0	37.5	37.5	−13.5	54 500
No. 11..........	1	0.0910	7.3	17.5	73.3	37.5	62 200
No. 10..........	2	0.1022	21.1	8.5	80.1	48.7	70 400
No. 10..........	4	0.1037	36.9	5.5	84.0	55.2	79 500
No. 10..........	6	0.1035	49.8	5.0	86.7	59.7	82 900
No. 16..........	0	0.0517	0	36.5	37.3	−16.7	55 100
No. 17..........	1	0.0460	10.6	21.5	69.1	30.1	61 300
No. 16..........	2	0.0516	22.3	7.5	78.7	45.5	69 500
No. 16..........	4	0.0503	39.5	3.5	84.5	56.2	79 700
No. 16..........	6	0.0512	49.8	4.0	86.3	59.0	82 800
No. 22..........	0	0.0257	0	32.0	36.4	....	54 400
No. 23..........	1	0.0301	7.8	20.0	66.6	26.1	60 600
No. 22..........	2	0.0266	17.2	7.0	75.5	41.1	69 100
No. 22..........	4	0.0263	36.1	2.5	81.4	51.0	79 600
No. 22..........	6	0.0263	48.7	2.0	83.3	55.0	83 700
No. 28..........	0	0.0128	0	35.5	23.0	....	54 500
No. 27..........	1	0.0145	11.5	16.0	65.6	....	62 800
No. 28..........	2	0.0132	21.2	4.5	72.2	....	69 200
No. 28..........	4	0.0137	35.6	1.5	77.1	....	80 700
No. 28..........	6	0.0130	47.4	1.0	81.1	....	84 900
No. 30..........	0	0.0105	0	30.5	....	....	54 600
No. 32..........	0	0.0085	0	32.0	....	....	55 100
No. 34..........	0	0.0059	0	28.5	....	....	55 600
No. 34..........	1	0.0077	12.5 17.6	6.0	....	....	65 200
No. 34..........	2	0.0058	25.0 29.4	2.5	....	....	74 400
No. 34..........	4	0.0062	41.2	1.0	....	....	78 800
No. 34..........	6	0.0060	50.8	1.0	....	....	87 900

NOTE.—Tensile strength and percentage elongation are an average of 3 specimens in each case; Rockwell hardness values are an average of 15 determinations of 5 readings on each of 3 tension test specimens.

28 per cent of zinc, gives the maximum tensile properties possible with the alpha brasses in the cold-rolled condition. A bar of alloy G brass of the chemical composition shown in Table V was rolled 2, 4, 6, 8 and 10 numbers hard in every even B. & S. gage number from No. 14 to No. 36, inclusive. Table VII gives the physical properties of the rolling series. The Rockwell hardness - tensile strength curve is shown in Fig. 6 and the tensile strength - reduction curve is shown by Fig. 7. The chemical composition limits are given in Table IV, and the tensile strength and Rockwell hardness limits are given in Table VIII.

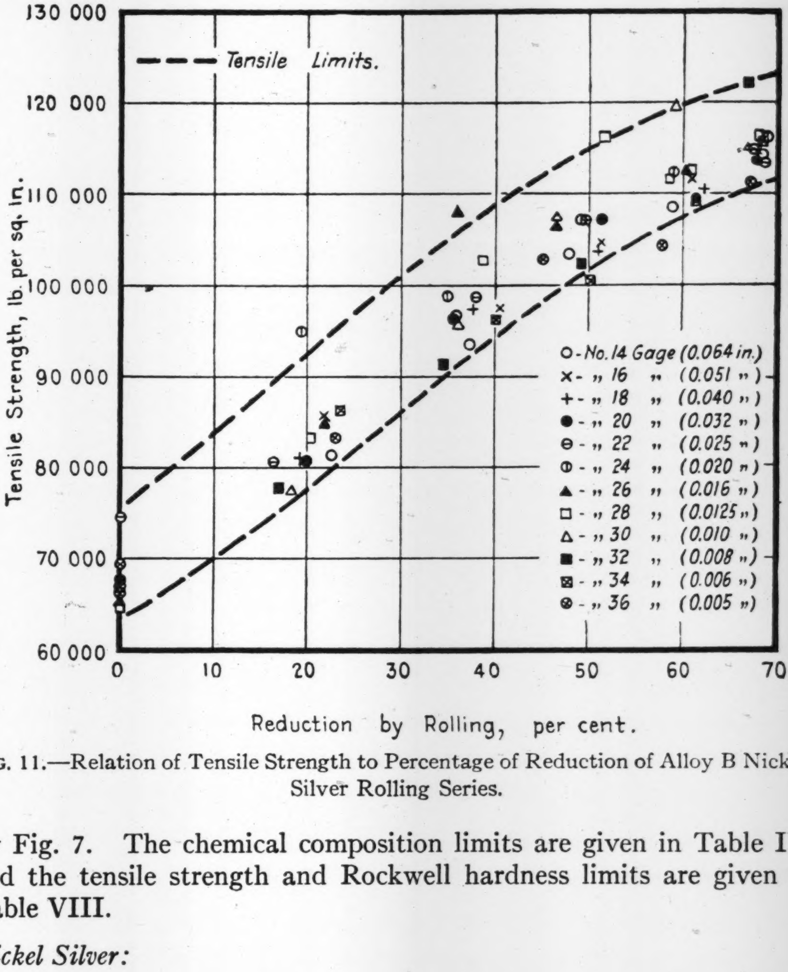

FIG. 11.—Relation of Tensile Strength to Percentage of Reduction of Alloy B Nickel-Silver Rolling Series.

Nickel Silver:

Two nickel-silver alloys were investigated, one containing nominally 72 per cent of copper, 10 per cent of zinc and 18 per cent of nickel, which will be designated "alloy A"; and the other containing 55 per cent of copper, 27 per cent of zinc, and 18 per cent of nickel and designated "alloy B." Alloy A is used mainly for forming and deep drawing purposes, whereas alloy B is used for springs.

Alloy A Nickel-Silver Sheet.—The average composition of three bars of alloy A cast from the same pot of metal are given in Table V. Table IX shows the physical properties of alloy A. Figure 8 shows the Rockwell hardness - tensile strength relationship and Fig. 9 shows the tensile strength limits plotted against percentage of reduction. The tensile strength and Rockwell hardness limits for this material are given in Table X and the composition limits in Table IV.

TABLE X.—ROCKWELL HARDNESS AND TENSILE STRENGTH LIMITS FOR ALLOYS A AND B NICKEL-SILVER SHEET.

Thickness	Temper, B. & S. Numbers Hard	Nominal Per-centage Reduc-tion by Rolling	Alloy A				Alloy B			
			Tensile Strength, lb. per sq. in.		Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in. Ball, 150-kg. Load (Red Figures)		Tensile Strength, lb. per sq. in.		Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in. Ball, 150-kg. Load (Red Figures)	
			Min-imum	Max-imum	Min-imum	Max-imum	Min-imum	Max-imum	Min. imum	Max-imum
0.040 in. and over	Quarter hard.... 1	11.0	58 500	67 000	26	47	......	......	..	..
0.020 to 0.040 in.		11.0	58 500	67 000	19	40	......	......	..	..
0.040 in. and over	Half hard.... 2	20.7	65 500	73 500	44	54	78 000	93 000	54	70
0.020 to 0.040 in.		20.7	65 500	73 500	38	48	78 000	93 000	48	67
0.040 in. and over	Hard.... 4	37.1	75 000	82 000	54	61	92 000	106 500	69	77
0.020 to 0.040 in.		37.1	75 000	82 000	49	55	92 000	106 500	66	75
0.040 in. and over	Extra hard.... 6	50.0	81 000	88 000	59	64	102 000	115 000	75	82
0.020 to 0.040 in.		50.0	81 000	88 000	54	60	102 000	115 000	72	79
0.040 in. and over	Spring.. 8	60.5	......	......	..	..	108 000	120 000	78	84
0.020 to 0.040 in.		60.5	......	......	..	..	108 000	120 000	75	81
0.040 in. and over	Extra spring...10	68.7	......	......	..	..	111 000	123 000	80	85
0.020 to 0.040 in.		68.7	......	......	..	..	111 000	123 000	77	82

Alloy B Nickel-Silver Sheet.—The five bars of alloy B had the average analysis given in Table V. The rolling series made from these five bars consisted of the tempers 2, 4, 6, 8 and 10 numbers hard for each of the even B. & S. gage numbers from No. 14 to No. 36, inclusive. The physical properties of this material are given in Table XI. The Rockwell hardness - tensile strength relationship is shown by Fig. 10 and the tensile strength limits plotted against percentage reduction are shown in Fig. 11. Table X shows the Rockwell hardness and tensile strength limits for alloy B. The chemical composition limits are given in Table IV.

Referring to the curve plotted in Fig. 11, showing the tensile strength limits plotted against percentage of reduction, it is seen that one series lies close to the upper curve whereas the remaining rolling

TABLE XI.—PHYSICAL PROPERTIES OF ALLOY B NICKEL-SILVER SHEET.

B. & S. Gage	Temper, B. & S. Numbers Hard	Thickness, in.	Actual Percentage Reduction by Rolling	Tensile Strength, lb. per sq. in.	Elongation in 2 in., per cent	Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in. Ball (Red Figures)	
						100-kg. Load	150-kg. Load
No. 14..........	0	0.0666	0	66 100	45.5	53.6	9.5
No. 14..........	2	0.0653	22.6	81 300	13.0	87.6	62.2
No. 14..........	4	0.0664	37.2	93 400	5.0	92.8	70.5
No. 14..........	6	0.0670	47.8	103 400	3.0	96.0	76.2
No. 14..........	8	0.0659	58.8	108 600	3.5	97.9	69.0
No. 14..........	10	0.0654	68.3	114 300	3.8	99.1	82.3
No. 16..........	0	0.0513	0	65 100	47.0	51.1	5.3
No. 16..........	2	0.0522	21.6	85 600	12.0	89.6	65.4
No. 16..........	4	0.0506	40.1	97 400	3.0	93.8	70.6
No. 16..........	6	0.0513	51.3	104 500	3.0	96.6	75.3
No. 16..........	8	0.0500	60.8	111 400	3.0	98.9	79.3
No. 16..........	10	0.0510	67.6	114 500	3.0	99.7	80.9
No. 18..........	0	0.0418	0	65 300	45.5	50.6	7.4
No. 18..........	2	0.0413	19.4	81 100	16.0	85.3	59.0
No. 18..........	4	0.0415	37.6	97 400	4.0	93.4	70.7
No. 18..........	6	0.0415	50.9	103 600	2.5	94.9	73.4
No. 18..........	8	0.0398	62.1	110 400	2.0	96.8	77.2
No. 18..........	10	0.0419	66.8	115 000	2.5	98.9	79.1
No. 20..........	0	0.0326	0	67 600	44.0	56.2	14.7
No. 20..........	2	0.0335	19.9	80 700	16.5	85.0	59.6
No. 20..........	4	0.0330	35.4	96 300	3.0	90.9	69.9
No. 20..........	6	0.0325	51.4	107 200	2.0	94.6	74.9
No. 20..........	8	0.0328	61.4	109 500	2.0	95.3	76.3
No. 20..........	10	0.0339	67.8	113 600	2.0	96.7	77.8
No. 22..........	0	0.0286	0	74 600	35.5	67.6	34.5
No. 22..........	2	0.0271	16.5	80 600	19.5	83.4	57.2
No. 22..........	4	0.0259	37.8	98 700	2.0	91.3	70.1
No. 22..........	6	0.0260	49.6	107 000	1.5	94.0	74.2
No. 22..........	8	0.0262	60.6	112 200	1.5	95.5	76.6
No. 22..........	10	0.0264	68.7	113 500	2.0	96.0	77.3
No. 24..........	0	0.0209	0	66 900	42.0	60.6	19.0
No. 24..........	2	0.0210	19.4	94 900	8.5	90.6	68.8
No. 24..........	4	0.0211	34.9	98 700	2.0	90.9	69.0
No. 24..........	6	0.0208	49.1	107 000	1.0	93.4	73.2
No. 24..........	8	0.0211	58.8	112 400	1.5	95.5	76.1
No. 24..........	10	0.0209	68.9	116 200	1.5	96.6	78.5
No. 26..........	0	0.0177	0	65 200	41.0	57.9	2.7
No. 26..........	2	0.0166	21.9	83 000	11.5	86.3	58.7
No. 26..........	4	0.0167	35.8	108 200	2.0	94.2	72.6
No. 26..........	6	0.0172	40.3	106 600	1.5	93.3	71.9
No. 26..........	8	0.0166	60.2	112 600	1.0	95.4	74.8
No. 26..........	10	0.0170	67.4	114 700	1.0	96.2	77.4
No. 28..........	0	0.0139	0	64 500	34.0	59.8	3.3
No. 28..........	2	0.0136	20.3	83 300	12.5	84.9	50.3
No. 28..........	4	0.0126	38.5	102 800	1.5	92.4	63.8
No. 28..........	6	0.0128	51.4	116 300	1.5	96.3	72.7
No. 28..........	8	0.0133	58.4	111 500	1.0	95.4	73.3
No. 28..........	10	0.0133	68.0	116 400	1.0	96.3	74.7
No. 30..........	0	0.0106	0	65 300	39.0	56.2	....
No. 30..........	2	0.0111	18.4	77 500	14.5	80.6	....
No. 30..........	4	0.0112	35.9	95 600	2.5	89.2	....
No. 30..........	6	0.0105	46.4	107 500	1.0	94.0	....
No. 30..........	8	0.0107	59.1	119 500	1.0	97.1	....
No. 30..........	10	0.0106	66.7	115 100	1.0	96.7	....
No. 32..........	0	0.0092	0	67 000	38.0	55.1	....
No. 32..........	2	0.0087	17.0	76 800	16.0	71.5	....
No. 32..........	4	0.0089	34.6	91 200	2.0	83.0	....
No. 32..........	6	0.0092	49.1	102 400	1.0	90.4	....
No. 32..........	8	0.0084	60.5	112 600	1.0	93.4	....
No. 32..........	10	0.0086	66.9	122 200	1.0	97.6	....
No. 34..........	0	0.0070	0	66 200	33.0	66.4	....
No. 34..........	2	0.0068	23.5	86 200	6.5	75.7	....
No. 34..........	4	0.0066	40.0	96 300	2.0	79.1	....
No. 34..........	6	0.0069	50.0	100 500	1.0	81.0	....
No. 34..........	8	0.0068	61.1	109 200	1.0	83.3	....
No. 34..........	10	0.0067	68.3	115 900	1.0	87.2	....
No. 36..........	0	0.0055	0	69 400	34.0	78.9	....
No. 36..........	2	0.0063	23.0	83 200	8.5	81.5	....
No. 36..........	4	0.0058	35.3	96 500	1.5	81.6	....
No. 36..........	6	0.0055	45.0	102 700	1.0	83.1	....
No. 36..........	8	0.0059	57.7	104 100	1.0	82.9	....
No. 36..........	10	0.0057	67.1	111 100	1.0	84.9	....

NOTE.—Tensile strength values are an average of 3 specimens in each case; Rockwell hardness values are an average of 15 determinations of 5 readings on each of 3 tension test specimens.

series are nearer to the lower curve. This is due to the fact that the bar rolled to No. 22 B. & S. gage sheet had a lighter anneal in the ready-to-finish condition than the other bars, but it was felt that this anneal was close to the commercial range of annealing practice and consequently this series was given some weight when the commercial limits were drawn up.

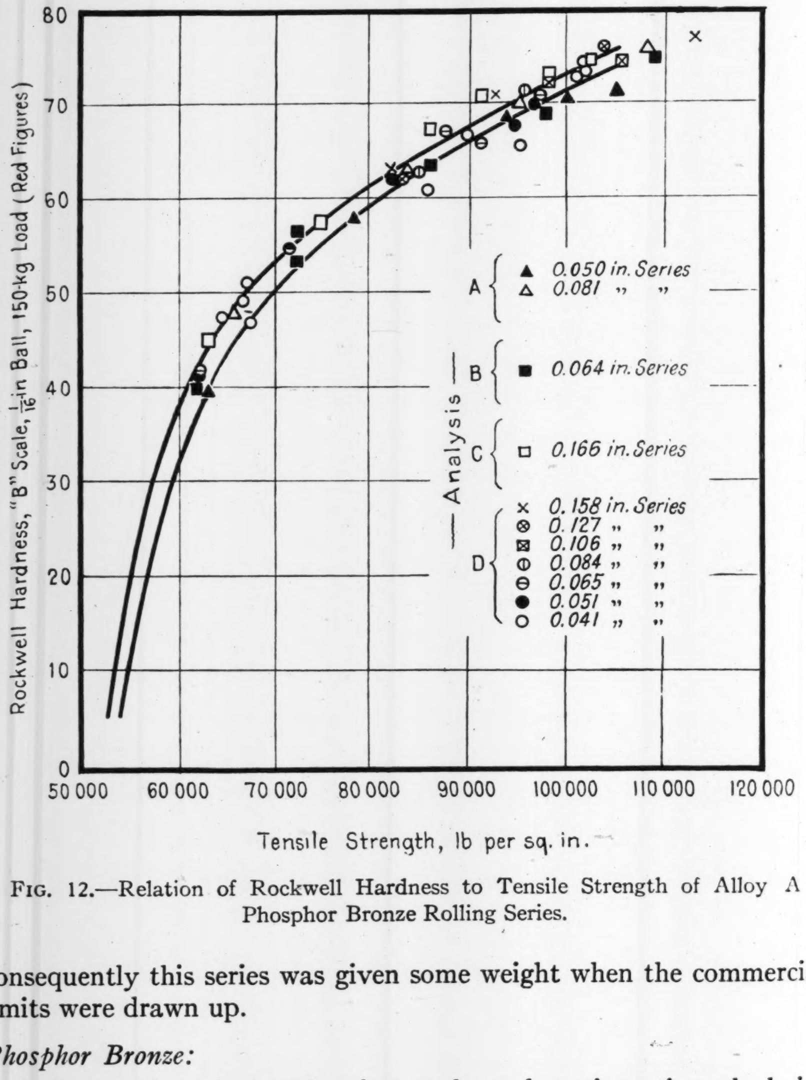

FIG. 12.—Relation of Rockwell Hardness to Tensile Strength of Alloy A Phosphor Bronze Rolling Series.

Phosphor Bronze:

Two grades of phosphor bronze have been investigated, designated alloys A and C, containing 4 and 8 per cent of tin, respectively.

Alloy A Phosphor-Bronze Sheet.—Three rolling series representative of the entire number investigated were made from the bar of metal of the analysis shown in Table V. These three series were rolled to all even numbers hard from 2 to 10, B. & S. gage for Nos. 14, 20 and 28. The physical properties are given in Table XII. The Rockwell hardness - tensile strength relationship for alloy A is given in Fig. 12 and the tensile strength - reduction relationship is shown in Fig. 13. The Rockwell hardness and tensile strength limits for alloy A are given in Table XIII and the chemical composition requirements are given in Table IV.

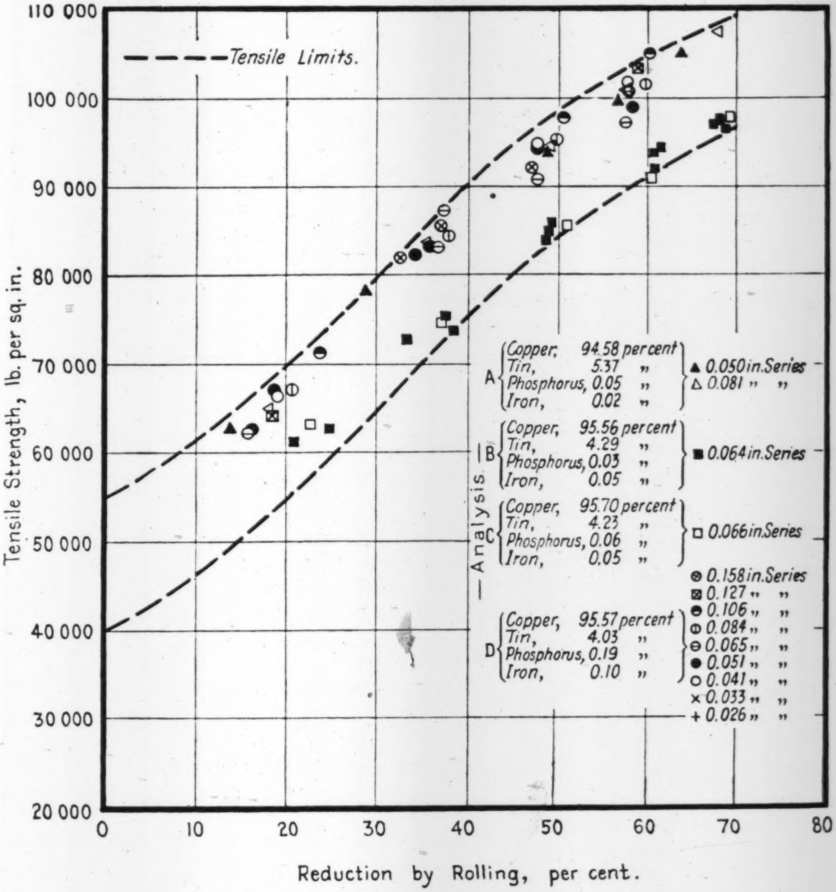

FIG. 13.—Relation of Tensile Strength to Percentage of Reduction by Rolling of Alloy A Phosphor Bronze Rolling Series

Alloy C Phosphor-Bronze Sheet.—The alloy C rolling series were made from five bars of metal having the average composition shown in Table V. This material was rolled to all even hardness numbers from 2 to 10 numbers hard, for all even B. & S. gage numbers from

No. 14 to No. 36, inclusive. The physical properties of this material are given in Table XIV. The Rockwell hardness - tensile strength relationship is shown in Fig. 14 and the tensile strength - reduction relationship is shown in Fig. 15. The physical requirements for alloy C are given in Table XIII and the chemical requirements in Table IV.

TABLE XII.—PHYSICAL PROPERTIES OF ALLOY A PHOSPHOR-BRONZE SHEET.

Temper, B. & S. Numbers Hard	B. & S. Gage, in.	Actual Percentage Reduction by Rolling	Tensile Strength, lb. per sq. in.	Elongation in 2 in., per cent	Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in. Ball (Red Figures)	
					100-kg. Load	150-kg. Load
Soft	0.0636	0	46 900	53	23.8	−6.9
2	0.0501	21.2	61 200	21.8	75.9	43.3
4	0.0395	37.8	75 150	8.0	84.8	58.3
6	0.0321	49.5	85 700	4.0	88.9	65.2
8	0.0248	61.0	93 900	2.5	91.4	68.8
10	0.0203	68.1	97 450	2.0	92.2	69.7
Soft	0.0324	0	47 000	51	26.6	−7.6
2	0.0242	25.2	62 500	15.5	75.3	43.2
4	0.0199	38.6	73 600	6	80.6	48.0
6	0.0166	48.8	83 950	2.5	84.4	55.0
8	0.0126	61.0	91 900	1.5	88.1	....
10	0.0105	67.6	97 350	1.5	89.3	....
Soft	0.0162	0	46 350	38.5	....	....
2	0.0129	20.4	61 250	15.5	....	....
4	0.0107	33.3	72 800	4.5	....	....
6	0.0082	49.2	84 900	1.5	....	....
8	0.0062	61.7	94 450	1.3	....	....
10	0.0050	68.8	96 400	1	....	....

NOTE.—Tensile strength and percentage elongation are an average of 3 specimens in each case; Rockwell hardness values are an average of 15 determinations of 5 readings on each of 3 tension test specimens.

TABLE XIII.—ROCKWELL HARDNESS AND TENSILE STRENGTH LIMITS FOR ALLOYS A AND C, PHOSPHOR-BRONZE SHEET.

Thickness	Temper, B. & S. Numbers Hard	Nominal Percentage Reduction by Rolling	Alloy A Phosphor Bronze				Alloy C Phosphor Bronze			
			Tensile Strength, lb. per sq. in.		Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in. Ball, 150 kg. Load (Red Figures)		Tensile Strength, lb. per sq. in.		Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in. Ball, 150 -kg. Load (Red Figures)	
			Minimum	Maximum	Minimum	Maximum	Minimum	Maximum	Minimum	Maximum
0.040 in. and over	Half hard.... 2	20.7	55 000	70 000	20	53	69 000	84 000	47	68
0.020 to 0.040 in.		20.7	55 000	70 000	15	51	69 000	84 000	40	64
0.040 in. and over	Hard.... 4	37.1	72 000	87 000	55	66	85 000	100 000	69	77
0.020 to 0.040 in.		37.1	72 000	87 000	53	64	85 000	100 000	65	74
0.040 in. and over	Extra hard.... 6	50.0	84 000	98 500	64	73	97 000	111 500	76	82
0.020 to 0.040 in.		50.0	84 000	98 500	62	71	97 000	111 500	73	79
0.040 in. and over	Spring.. 8	60.5	91 000	105 000	69	75	105 000	118 500	79	85
0.020 to 0.040 in.		60.5	91 000	105 000	67	73	105 000	118 500	76	82
0.040 in. and over	Extra spring...10	68.7	96 000	109 000	72	77	109 500	122 000	81	86
0.020 to 0.040 in.		68.7	96 000	109 000	70	75	109 500	122 000	78	83

TABLE XIV.—PHYSICAL PROPERTIES OF ALLOY C PHOSPHOR-BRONZE SHEET.

B & S. Gage	Temper, B. & S. Numbers Hard	Thickness, in.	Actual Percentage Reduction by Rolling	Tensile Strength, lb. per sq. in.	Elongation in 2 in., per cent	Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in. Ball (Red Figures)	
						100-kg. Load	150-kg. Load
No. 14	0	0.0663	0	62 400	68.0	56.3	14.7
No. 14	2	0.0640	22.0	72 800	36.0	83.7	56.0
No. 14	4	0.0649	37.5	86 900	17.0	93.0	71.3
No. 14	6	0.0659	50.5	100 900	10.0	97.3	78.3
No. 14	8	0.0671	59.4	110 300	7.5	99.6	81.6
No. 14	10	0.0669	67.7	116 300	6.5	101.4	84.7
No. 16	0	0.0515	0	54 700	76.0	42.0	−7.3
No. 16	2	0.0536	19.4	80 400	33.0	88.4	63.6
No. 16	4	0.0539	35.4	83 800	20.0	91.9	67.7
No. 16	6	0.0519	49.8	97 100	9.0	96.4	76.2
No. 16	8	0.0535	60.2	107 700	7.0	99.2	80.6
No. 16	10	0.0549	67.1	115 700	7.0	101.1	83.2
No. 18	0	0.0429	0	60 400	71.0	50.7	7.7
No. 18	2	0.0412	20.6	71 200	36.0	82.3	55.0
No. 18	4	0.0430	34.9	99 200	11.5	96.0	73.4
No. 18	6	0.0431	48.2	95 700	10.0	94.5	73.4
No. 18	8	0.0420	59.9	106 500	5.5	97.7	78.4
No. 18	10	0.0416	69.3	116 800	4.0	100.2	82.6
No. 20	0	0.0336	0	61 200	65.5	50.2	6.7
No. 20	2	0.0344	18.6	78 300	34.0	85.4	60.3
No. 20	4	0.0332	35.6	87 500	15.0	91.0	69.1
No. 20	6	0.0334	49.4	114 200	5.0	98.3	80.4
No. 20	8	0.0347	57.9	105 900	5.0	96.1	77.1
No. 20	10	0.0332	68.5	113 600	3.0	98.6	80.7
No. 22	0	0.0264	0	53 700	74.0	45.8	−0.4
No. 22	2	0.0267	22.0	80 200	32.0	85.4	60.8
No. 22	4	0.0273	35.5	95 400	12.0	92.9	72.8
No. 22	6	0.0263	49.5	102 400	7.0	94.6	75.1
No. 22	8	0.0260	61.3	121 000	3.0	99.0	82.9
No. 22	10	0.0266	67.7	112 100	3.0	97.3	80.3
No. 24	0	0.0221	0	59 700	67.5	52.3	11.0
No. 24	2	0.0200	22.4	77 300	26.0	85.8	60.2
No. 24	4	0.0219	35.2	95 500	14.0	92.2	71.4
No. 24	6	0.0221	48.8	108 500	5.0	95.9	77.1
No. 24	8	0.0213	55.1	110 700	3.0	96.9	78.8
No. 24	10	0.0216	67.4	124 800	2.0	100.0	84.0
No. 26	0	0.0174	0	54 300	69.0	57.3	25.0
No. 26	2	0.0176	22.6	73 600	38.0	81.5	52.6
No. 26	4	0.0178	32.9	86 800	15.0	89.2	65.9
No. 26	6	0.0166	49.9	108 500	4.0	96.1	76.3
No. 26	8	0.0173	60.1	116 600	2.0	97.3	78.7
No. 26	10	0.0168	65.7	115 700	2.0	97.7	80.3
No. 28	0	0.0130	0	57 500	58.0	61.6	26.5
No. 28	2	0.0129	23.2	73 400	30.0	82.4	46.4
No. 28	4	0.0137	40.1	91 500	11.0	90.8	66.1
No. 28	6	0.0128	52.9	105 300	4.0	95.1	69.6
No. 28	8	0.0150	55.6	107 000	3.0	95.1	74.1
No. 28	10	0.0138	66.4	117 600	1.0	99.0	81.1
No. 30	0	0.0110	0	55 400	61.0	65.3	43.1
No. 30	2	0.0110	17.6	72 400	27.5	85.2	50.3
No. 30	4	0.0111	34.2	86 500	12.5	90.3	54.2
No. 30	6	0.0109	53.0	102 200	2.5	94.5	63.1
No. 30	8	0.0115	58.8	101 900	1.0	96.1	69.7
No. 30	10	0.0114	66.7	111 000	1.5	98.3	74.4
No. 32	0	0.0086	0	58 200	52.0	....	....
No. 32	2	0.0077	28.6	89 000	8.0	....	....
No. 32	4	0.0087	28.6	91 400	9.0	....	....
No. 32	6	0.0090	47.6	97 200	3.0	....	....
No. 32	8	0.0088	61.3	108 200	1.5	....	....
No. 32	10	0.0088	67.0	115 300	1.5	....	....
No. 34	0	0.0063	0	63 900	50.0	....	....
No. 34	2	0.0063	24.0	81 900	17.5	....	....
No. 34	4	0.0061	41.9	99 100	7.0	....	....
No. 34	6	0.0063	44.5	111 300	2.5	....	....
No. 34	8	0.0064	63.4	110 200	1.5	....	....
No. 34	10	0.0066	70.0	118 800	1.0	....	....
No. 36	0	0.0054	0	61 300	51.0	....	....
No. 36	2	0.0054	16.1	77 900	25.0	....	....
No. 36	4	0.0049	35.4	97 900	4.5	....	....
No. 36	6	0.0053	52.4	103 000	4.0	....	....
No. 36	8	0.0054	58.0	114 800	1.5	....	....
No. 36	10	0.0059	68.3	110 500	1.5	....	....

NOTE.—Tensile strength and percentage elongation are an average of 3 specimens in each case; Rockwell hardness values are an average of 15 determinations of 5 readings on each of the 3 tension test specimens.

EXPERIENCE DATA

The collection of experience data as described in the previous paper was continued on commercial shipments of the sheet materials under discussion. The shipments ranged in size from 100 to about 50,000 lb. of sheet metal and included material from at least five different mills. The data shown comprise Rockwell hardness and tensile strength values which are the averages of tests on individual shipments.

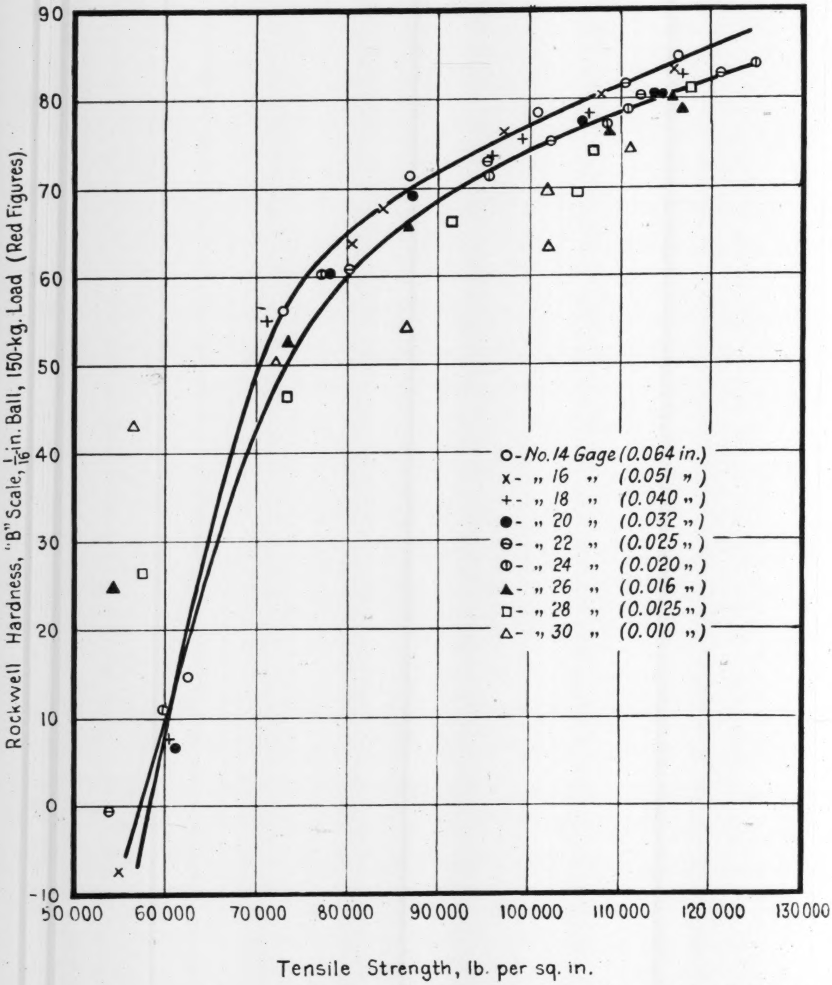

FIG. 14.—Relation of Rockwell Hardness to Tensile Strength of Alloy C Phosphor Bronze Rolling Series.

Figure 16 shows graphically the data obtained on high and clock brass sheet, and Fig. 17 the experience data which have been averaged and shown as a smooth curve for comparison with the specification limits shown as points for high brass sheet. The agreement is close. Figures 18, 19 and 20 show similar smooth curves of experience data for nickel silver and phosphor bronze. In these latter cases the experience data are not sufficiently complete to verify the specification limits in all tempers, but in general the accuracy of the specification limits is verified.

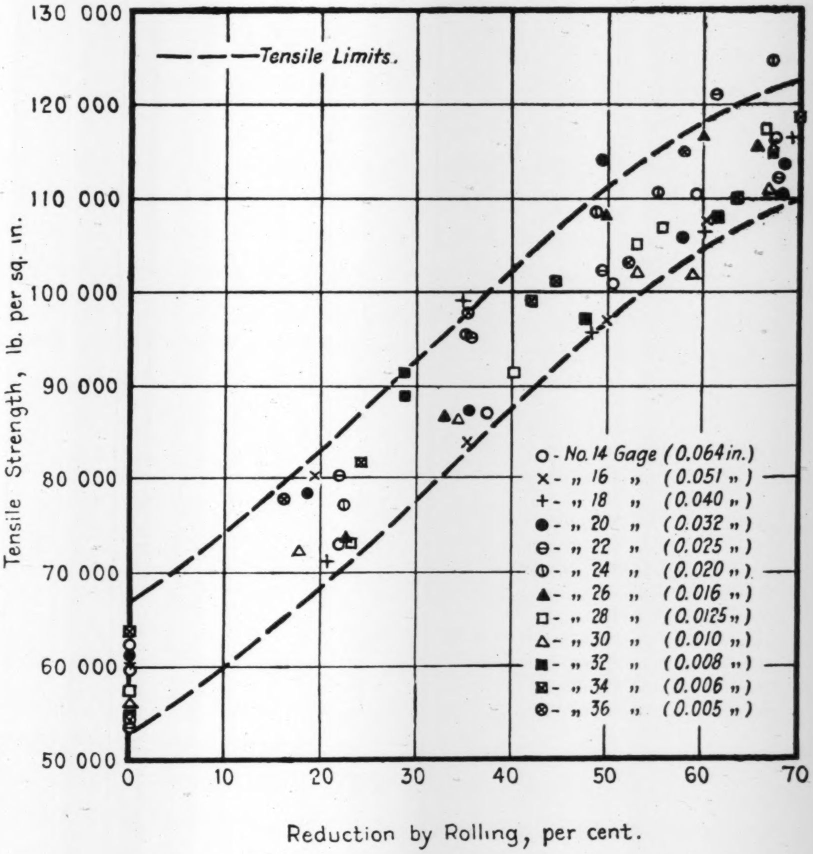

FIG. 15.—Relation of Tensile Strength to Percentage of Reduction of Alloy C Phosphor Bronze Rolling Series.

In addition to the data, the experience of mills supplying material indicates that the limits proposed are satisfactory. Instances where the limits were not met served only to emphasize the advantages of control of the physical properties. Variations from the nominal annealing temperature, composition or percentage reduction in all cases explained the variations from the specification limits.

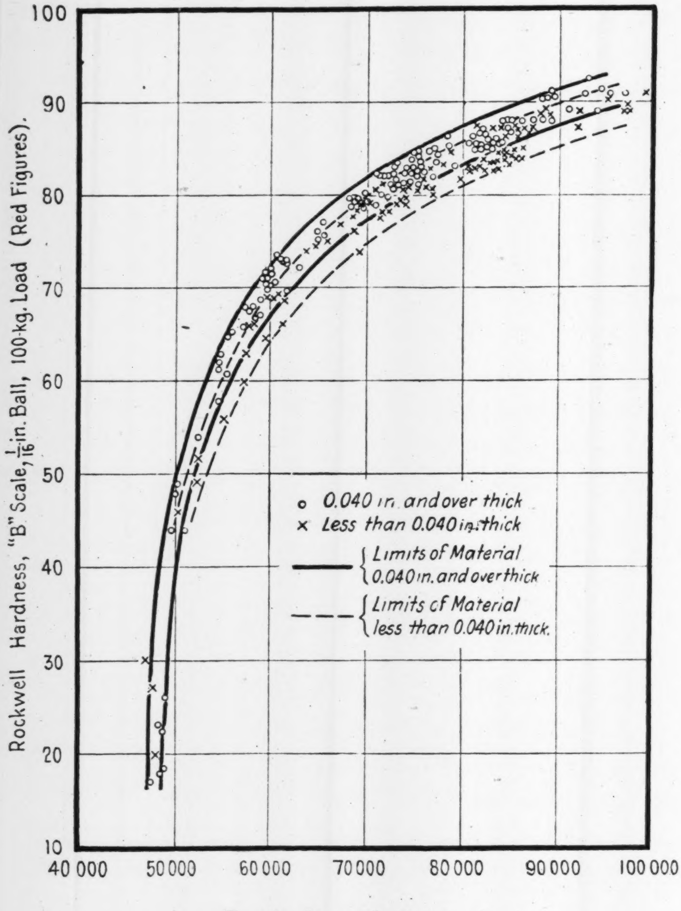

FIG. 16.—Accumulated Data on High and Clock Brass Sheet.

CONCLUSION

The purposes stated in this and the former paper have been accomplished, namely, the development of a commercial method of test for sheet non-ferrous metals and limits for use in commercial specifications.

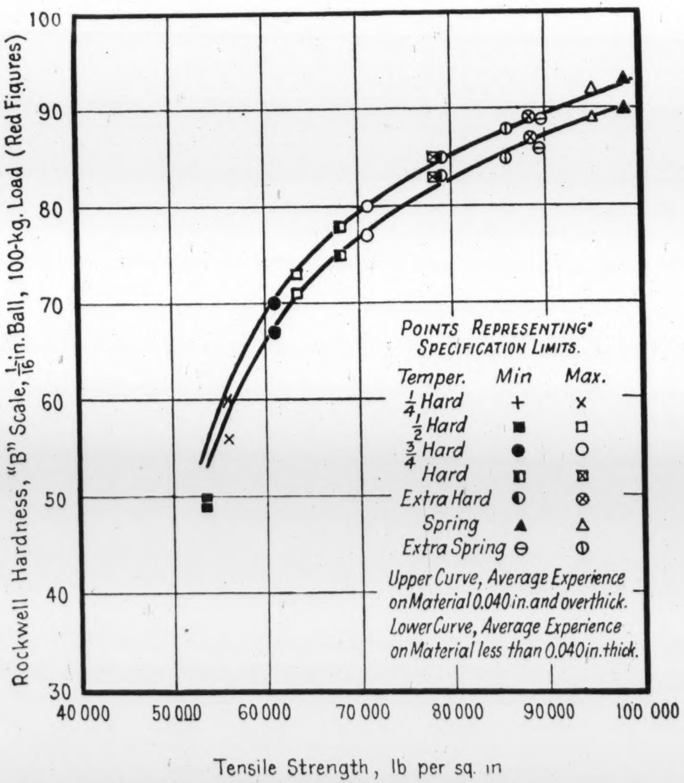

FIG. 17.—Comparison of Curves of Results from Experience Data with Limits Adopted, for High Brass Sheet.

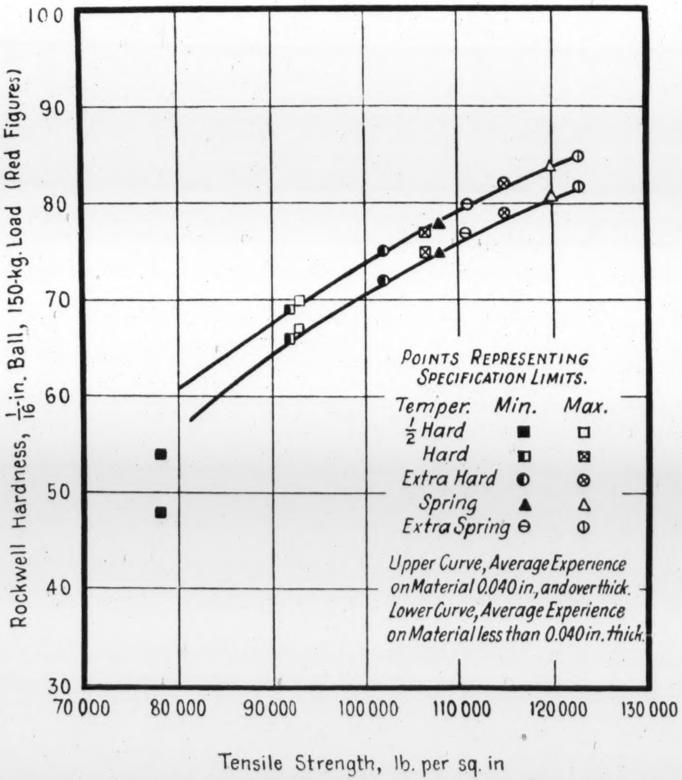

Fig. 18.—Comparison of Curves of Results from Experience Data with Limits Adopted, for Alloy B Nickel-Silver Sheet.

The conclusion reached in the former paper[1] that the tension test is the best available static test for non-ferrous sheet metal in the hard tempers has been confirmed. A number of other physical tests were investigated and while they all showed usefulness they were not considered satisfactory for inclusion in specifications.

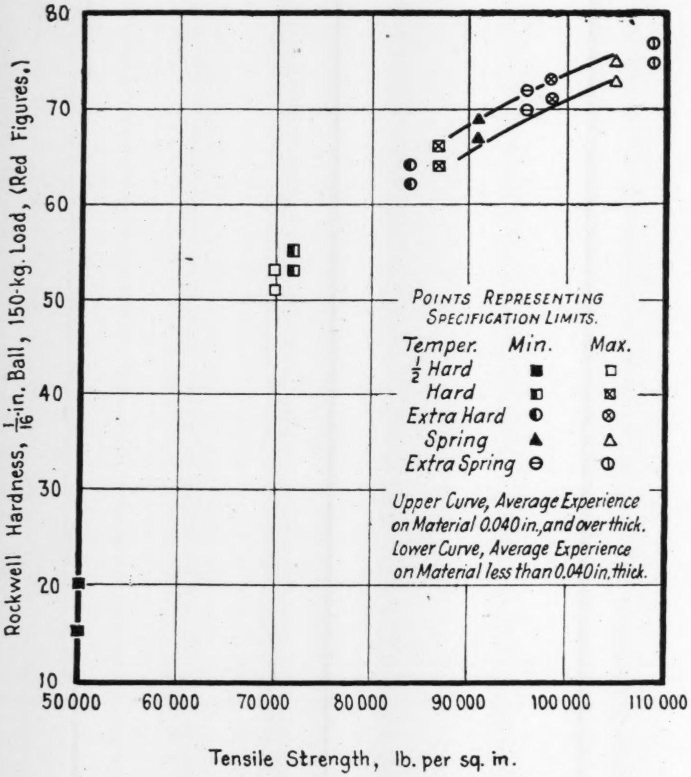

FIG. 19.—Comparison of Curves of Results from Experience Data with Limits Adopted, for Alloy A Phosphor-Bronze Sheet.

Because of its close correlation to the tension test, the Rockwell hardness test has been adopted for preliminary inspection purposes. Other hardness tests have been studied but this test is the most satisfactory for sheet metals 0.020 in. thick and thicker. It has limited use for material less than 0.020 in. thick. Material near to or outside of the Rockwell hardness limits is subjected to the tension test. Rejection of material is based solely on the tension test.

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[1] Van Deusen, Shaw and Davis, *loc. cit.*

Refinements in the application of the Rockwell hardness test as an inspection instrument have been worked out and it is shown that agreements of $1\frac{1}{2}$ Rockwell numbers or better can be obtained between producer and consumer on commercial shipments of material.

The necessity for the adoption of a series of Rockwell standard blocks covering the range of hardness of non-ferrous sheet and the calibration of testing machines to these standards is emphasized.

Grain-size limits are given for annealed brass of four compositions and for one nickel-silver alloy. This is the most satisfactory method of controlling the annealed material.

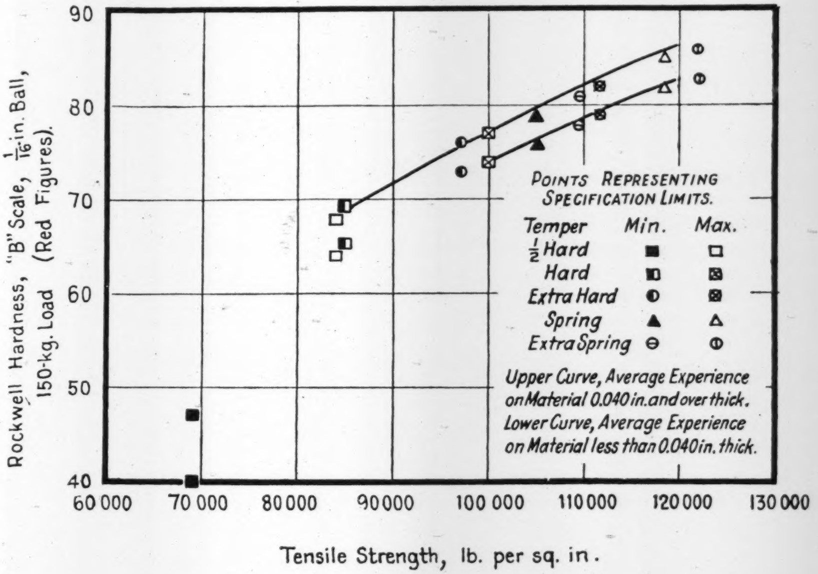

FIG. 20.—Comparison of Curves of Results from Experience Data with Limits Adopted, for Alloy C Phosphor-Bronze Sheet.

Tentative tensile strength and Rockwell hardness limits for alloy G brass, nickel-silver and phosphor-bronze alloys have been developed. These limits are considered preliminary until more complete information on actual shipments of material is available. The limits for high and clock brass described in the previous paper as tentative are now considered entirely satisfactory with the modifications proposed in this paper.

A modified bend test has been presented as having considerable value in determining the forming and drawing qualities of sheet metals in the annealed condition.

DISCUSSION

MR. J. L. CHRISTIE[1] (*presented in written form*).—This paper is a welcome addition to the published data on non-ferrous sheet metal. Its sources make it authoritative. The value of such dissemination of information can be best appreciated by those of us who are in the brass industry, for it was not so long ago that consumers quite evidently were intentionally misled as to what they were receiving from their source of supply, so that if they ordered the material from another source and received what they specified, they would surely run into trouble. Very few users of brass sheet know what they want in terms of the metal itself, let alone how to specify and test it; they simply know what it must do.

The completeness of the work leaves room for few comments.

We endorse the conclusion that the grain size measurement is the best way of controlling annealed material. It is the practice at the Bridgeport Brass Co. to test with the Rockwell machine a relatively large number of samples from a given lot, and then to make grain size determinations on the hardest and softest.

I am much interested in Table II of the paper which gives the average grain diameter for annealed brass and nickel silver. It has been our experience that a great many users of soft sheet brass cannot tolerate the wide limits given for each of the three anneals for any metal. For instance, an article made by simply cupping and forming high brass or alloy E would not turn out to be satisfactory if the metal covered the entire grain size range of either the light or drawing anneal. Metal with a 0.035-mm. grain size might be satisfactory but metal with a 0.010-mm. grain size would be too hard, and metal with a 0.090-mm. grain size would give too rough a surface. I realize these remarks refer more to the standards adopted than to the method of testing.

Table VI gives values for the modulus of elasticity. I should like to inquire if there is any reason for the seeming lack of consistency of the values given?

MR. H. L. WHITTEMORE[2] (*presented in written form*).—Experience at the Bureau of Standards with indentation testing machines, such as the Brinell, Rockwell, and Vickers, have led to the conclusion that the only satisfactory way of securing permanent standards for the readings of such machines is to base the standardization directly on the fundamental standards of force, length, and time.

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[1] Metallurgist, Bridgeport Brass Co., Bridgeport, Conn.

[2] Chief, Engineering Mechanics Section, VI-5, U. S. Bureau of Standards, Washington, D. C.

We, in common with other laboratories, have found with Brinell machines that if, within proper tolerances, the diameter of the ball, the applied load, the rate of loading and duration of loading are maintained correct, the Brinell hardness readings on a reasonably uniform block can be repeated after an interval of years and check with the average of the surrounding impressions.

As Townsend, Straw and Davis have found with their Rockwell blocks, it is not easy to obtain test blocks that will not show a variation in Brinell hardness number, so that the only way we have been able to secure checks of this kind within the limits to be expected from the tolerances set on load, ball diameter, and time has been to make an indentation map of the surface of each block. Only when the contour lines of equal Brinell hardness number are nearly straight and not too closely spaced does the diameter of the impression on a "standard" block furnish as close a check on the condition of the machine as the fundamental calibration in terms of force, length, and time.

There seems to be no necessary reason why the principle first worked out by Devries[1] at the Bureau of Standards of measuring the difference of height of the indenting tool under a minor load, before and after the application of the major load, which is commercially embodied in the Rockwell machine, should not permit of a similar standardization on the fundamental basis of force, length and time.

No blocks standardized arbitrarily by the manufacturers of the machine are suitable standards to be maintained by the Bureau of Standards when a standardization on the basis of the fundamental units of mass, length, and time is possible.

If, as stated by Townsend, Straw, and Davis, "wear of the operating parts and slight variations in friction" produce intolerable variations in their "standard machine" the answer is not to side step the real difficulty by setting up arbitrary standards but to provide means for checking and eliminating these variations in the Rockwell machine.

MR. D. K. CRAMPTON[2] (*presented in written form*).—In connection with the use of the Rockwell hardness tester, we should not lose sight of the fact that the division into gages of 0.020 to 0.040 in., and 0.040

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[1] R. P. Devries, "Comparison of Five Methods Used to Measure Hardness," U. S. Bureau of Standards *Technical Paper No. 11*.

[2] Metallurgist, Chase Metal Works, Waterbury, Conn,

in. and over, is only a first approximation although it is sufficient when considering rolled material. This same broad division cannot be applied to annealed material, as with such the hardness number varies markedly with the thickness of the sheet.

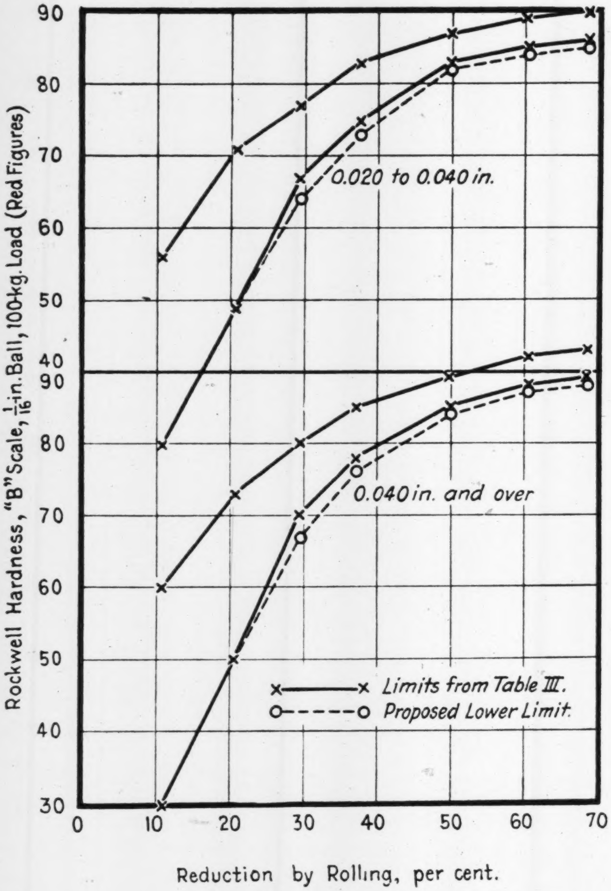

FIG. 1.—Relation of Rockwell Hardness to Reduction by Rolling.

It is noted that on page 391 the 150-kg. load is recommended for nickel silver and phosphor bronze alloys and the 60-kg. load used for testing of annealed material. We think this is as it should be; in fact, we have for several years strongly advocated the use of the 60-kg. load instead of the 100-kg. load in testing of annealed brasses.

It is noted that throughout the paper in referring to Rockwell hardness values the conditions of test are specified completely as for instance "Rockwell hardness, 'B' scale, $\frac{1}{16}$-in. ball, 100-kg. load (red figures)." We have found it extremely helpful in our laboratory to abbreviate this designation of conditions of test by giving in parentheses after the hardness value one of the following designations:

ABBREVIATED DESIGNATION	COMPLETE DESIGNATION
16–60–B	$\frac{1}{16}$-in. ball, 60-kg. load, "B" scale
16–100–B	$\frac{1}{16}$-in. ball, 100-kg. load, "B" scale
16–150–B	$\frac{1}{16}$-in. ball, 150-kg. load, "B" scale
C–150–C	Diamond cone, 150-kg. load, "C" scale
8–100–B	$\frac{1}{8}$-in. ball, 100-kg. load, "B" scale

In recording Rockwell values this abbreviated designation is invariably used after the hardness number and eliminates all confusion as to what conditions of test have been used. We would record, for instance, Rockwell 69 (16-60-B). It is thought that such an abbreviation might well be generally adopted as its advantages would seem obvious.

The tensile strength limits given in Table III for high, clock, and alloy E brass sheet check well with our own findings and we believe would form a proper basis for specification limits. The Rockwell hardness values given in the same table we do not check quite as nicely. Our experience has been that the curve of Rockwell hardness *versus* reduction is slightly flatter than that indicated by the given values and, further, that a somewhat greater range is necessary in tempers from 3 to 10 numbers hard, inclusive. We base our opinion on carefully controlled laboratory tests and a large number of mill control tests. The values we would recommend in place of those given in Table III are as follows:

THICKNESS	TEMPER, B.&S. NUMBERS HARD	ROCKWELL HARDNESS, 16–100–B MINIMUM	ROCKWELL HARDNESS, 16–100–B MAXIMUM
0.040 in. and over	1	30	60
0.020 in. to 0.040 in.	1	30	56
0.040 in. and over	2	50	73
0.020 in. to 0.040 in.	2	49	71
0.040 in. and over	3	67	80
0.020 in. to 0.040 in.	3	64	77
0.040 in. and over	4	76	85
0.020 in. to 0.040 in.	4	73	83
0.040 in. and over	6	84	89
0.020 in. to 0.040 in.	6	82	87
0.040 in. and over	8	87	92
0.020 in. to 0.040 in.	8	84	89
0.040 in. and over	10	88	93
0.020 in. to 0.040 in.	10	85	90

In the accompanying Fig. 1 there have been plotted the original data in solid lines and the proposed minimum limits in dotted lines.

Inasmuch as the tensile strength limits and Rockwell hardness limits even revised as proposed are fairly narrow for commercial production, it might not be amiss to say a word about the proper use of such specification limits. This is not meant as a specific criticism of the present paper but is equally applicable to all specification limits for similar materials. Frequently a user of such materials in determining the temper of metal required may have considerable doubt in his mind as to whether it would be better to use, say, 4 numbers hard or 6 numbers hard sheet. Having determined which of these two he will use, possibly by guess, or even by the toss of a coin, he then sets about defining his wants with meticulous accuracy. We would simply wish to caution users of material to use specification limits such as these in a judicious manner and not impose on the producer hardships which are not warranted by the use to which the material is to be put.

MR. R. J. WHEELER.[1]—As producers of certain of these materials, we are, of course, very much interested in this excellent paper. We have felt a real need for a commercial method of test, and it is our opinion that the tension test with the Rockwell hardness test for preliminary inspection, as outlined in this paper, is the most satisfactory. We have been using the Rockwell hardness test for controlling the tempers of phosphor bronze and nickel silver for three years with very satisfactory results.

I should like to ask the authors why the proposed tensile strength limits for phosphor bronzes were based upon alloys with a phosphorus content on the minimum allowed in the specifications as shown on pages 406 and 407, in the case of alloy A phosphor bronze sheet and alloy C phosphor bronze sheet, and as given in Table V. We have found that with a given tin content there is an increase in tensile stength when the phosphorus content is at or near the maximum allowed.

In connection with the Rockwell hardness test, I should also like to ask if the rate of speed in applying the minor load has any influence upon the uniformity of the readings obtained.

MESSRS. J. R. TOWNSEND[2], W. A. STRAW[3] and C. H. DAVIS[4] (*authors' closure by letter*).—With regard to the question of the effect of speed of applying the minor load of the Rockwell hardness tester as raised by Mr. Wheeler the authors feel that this is of minor inportance. The amount of the minor load is relatively small and the degree of penetration is not sufficient to cause wide discrepancies in the results. On the other hand, we feel that it would be wrong to apply the minor load suddenly or to over-run the mark on the gage stem which indicates when the minimum load has been applied.

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[1] Metallurgist, The Riverside Metal Co., Riverside N. J.

[2] Member Technical Staff, Bell Telephone Laboratories, Inc., New York City.

[3] Development Engineer, Western Electric Co., Inc., Chicago, Ill.

[4] Assistant Metallurgist, American Brass Co., Waterbury, Conn.

With regard to Mr. Crampton's remarks concerning a convenient method of abbreviating the Rockwell hardness scale it has been our practice to arrange the abbreviation in the following manner: Hardness number, scale, penetrator and load, for example, 52-B-$\frac{1}{16}$-100. This scheme of alternating the numbers and letters and fractions prevents the possibility of misunderstanding the hardness number which could result from the method described by Mr. Crampton.

With regard to Mr. Crampton's comment about the Rockwell - percentage reduction curve, the discrepancy between his results and ours may be explained as being due to the manner in which the tests were carried out. By reference to the paper it would be seen that our methods of test were carefully standardized before testing the material.

We agree with Mr. Whittemore that it would be preferable to standardize Rockwell test blocks and Rockwell machines on the basis of the standards of mass, length and time. This would be possible if the Rockwell hardness number were defined in terms of length, mass and time. This, however, is not the case and it is necessary, therefore, to have the next best thing in carrying out the standardization work and that is to agree on the hardness of standard blocks. This agreement was reached by taking the average of a number of Rockwell machines that were in correct adjustment. The use of these standard blocks in calibrating testing machines that have been used to inspect a large amount of material has amply demonstrated the value of this type of calibration.

The reason for the seeming lack of agreement in the determinations of modulus of elasticity as pointed out by Mr. Christie is a matter that has caused us some concern. The ripples and bows in the material which are caused by the almost impossibility of obtaining exact alignment of the rolls used for rolling the sheet metal tend to affect the load-elongation curve when the tension test specimens are tested. A paper presented by Mr. R. L. Templin at this annual meeting[1] develops these defects in considerable detail. We have listed the values as determined in the laboratories for these materials and have not attempted to round off the figures.

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[1] R. L. Templin, "The Determination and Significance of the Proportional Limit in the Testing of Metals," see p. 523.

HARDNESS RELATIONSHIPS AND PHYSICAL PROPERTIES OF SOME COPPER ALLOYS

By C. H. Davis[1] and E. L. Munson[2]

Synopsis

The hardness relationships of several commercial copper-zinc and copper-tin alloys, and three copper-nickel-zinc alloys have been studied. The evidence of tensile tests, and Brinell, Rockwell and scleroscope hardness values on annealing and rolling series has been assembled in tables of data and plotted in eleven charts.

The hardness - tensile strength relationship not only varies progressively with chemical composition, but is also dependent on the previous mechanical and thermal treatment to which any single alloy has been subjected. The hardness relationships (Brinell, Rockwell and scleroscope) are also determined by the governing conditions of composition and previous history of the material. This is likewise true of the tensile strength - elongation and hardness - elongation relationships.

A single line composite conversion chart is obviously liable to introduce considerable error, and the use of such curves should be discouraged.

The demonstration of relationships between hardness tests, and between these and tensile properties of materials, has heretofore been neither complete nor entirely clear.

The complexity of the subject has been recognized by some of the many investigators of hardness. Although known to a few, even these authors have not presented much that was concrete to substantiate their statements. Devries[3] noted from analytical considerations that hardness relations would be different for each metal under test. In a further study of hardness in its relation to other physical properties Devries[4] considers the matter further and quotes from Kürth[5] that no simple relation exists between the ultimate strength and the Brinell hardness of two pure metals. The senior author[6] has shown that a different hardness relationship existed between the scleroscope and Brinell tests on three copper-zinc alloys. In a short paragraph of a paper on cold-rolled copper Hoyt and Schermerhorn[1] observed, in the case of scleroscope, Rockwell and Brinell values on copper, that no one series of hardness numbers could be converted into another by using any single ratio. R. C. Brumfield[2] suggests that an accurate equation might be written for any given material by slightly altering the constants of the appropriate equation. In his paper there were not sufficient data to warrant this assumption. More recent studies of the relationships between hardness numbers [3,4,5] have not given sufficient consideration to individual alloys but have computed averages from tests on materials having a wide range of composition.

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[1] Assistant Metallurgist, American Brass Co., Waterbury, Conn.

[2] The American Brass Co., Waterbury, Conn.

[3] R. P. Devries, "A Comparison of Five Methods of Hardness Measurement," *Proceedings*, Am. Soc. Testing Mats., Vol. XI, p. 709 (1911).

[4] R. P. Devries, "Hardness in Its Relation to Other Physical Properties," *Proceedings*, Am. Soc. Testing Mats., Vol. XI, p. 726 (1911).

[5] A. Kürth, *Zeitschrift*, Verein Deutscher Ingenieure, Vol. 52, p. 873 (1908).

[6] C. H. Davis, "Testing of Sheet Brass," *Proceedings*, Am. Soc. Testing Mats., Vol. XVII, Part II, p. 164 (1917).

TABLE I.—ANALYSES OF MATERIALS USED IN THE TESTS.

	Copper, per cent	Zinc, per cent	Lead, per cent	Iron, per cent	Nickel, per cent	Manganese, per cent	Total Carbon, per cent	Phosphorus, per cent	Tin, per cent
Electrolytic Copper.....	99.952[a]	....	....	....	....	....	....	....	....
Copper-Zinc Alloys:									
Alloy A............	89.36	10.55	0.07	0.02	....	....	..	....	....
Alloy B............	85.60	14.35	0.01	0.04	....	....	....	....	....
Alloy C............	78.50	21.45	0.02	0.03	....	....	....	....	....
Alloy D............	71.73	28.22	0.02	0.03	....	....	....	....	....
Alloy E............	67.13	32.83	0.02	0.02	....	....	....	....	....
Alloy F............	66.30	33.64	0.02	0.04	....	....	....	....	....
Alloy G............	60.17	39.76	0.04	0.03	....	....	....	....	....
Nickel-Silver Alloys:									
Alloy H............	71.31	10.74	....	0.12	17.62	0.20	0.013	....	....
Alloy I............	63.97	17.77	....	0.11	17.92	0.20	0.029	....	....
Alloy J............	55.59	26.23	0.005	0.07	17.97	0.11	0.016	....	....
Copper-Tin Alloys:									
Alloy K............	95.57	....	0.02	0.10	....	....	....	0.19	4.03
Alloy L............	95.56	....	0.01	0.05	....	....	....	0.03	4.31
Alloy M............	95.50	....	....	....	....	....	....	0.26	3.96
Alloy N............	95.29	....	0.02	0.08	....	....	....	0.04	4.34
Alloy O............	94.64	....	....	0.03	....	....	....	0.08	5.11
Alloy P............	91.84	....	0.02	0.03	....	....	....	0.03	8.08
Alloy Q............	89.45	....	....	0.03	....	....	....	....	10.61

[a] Including silver.

An excellent paper by Dr. Otto Schwarz[6] on hardness relationships has been received since the preparation of the manuscript of the present paper.

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[1] S. L. Hoyt and T. R. Schermerhorn, "The Hardness of Cold Rolled Copper," *Journal*, Inst. of Metals, Vol. XXXV, p. 231.

[2] R. C. Brumfield, "Some Comparisons Between Rockwell and Brinell Hardness," *Transactions*, Am. Soc. Steel Treating, Vol. IX, No. 6, June, 1926, pp. 841–856.

[3] R. R. Moore, "Relationships Between Rockwell, Brinell, and Scleroscope Numbers," *Transactions*, Am. Soc. Steel Treating, Vol. XII, No. 6, December, 1927, p. 968.

[4] H. M. German, "Hardness Testing," *Transactions*, Am. Soc. Steel Treating, Vol. XIV No. 3, September, 1928, p. 343.

[5] S. N. Petrenko, "Relationships Between the Rockwell and Brinell Numbers," U. S. Bureau of Standards *Technologic Paper No. 334*.

[6] Otto Schwarz, "Zugfestigkeit und Härte bei Metallen," Forschungsarbeiten auf dem Gebiete des Ingenieurwesens, *Heft 313* (1929).

TABLE II.—TEST DATA ON COPPER AND COPPER ALLOYS.

	Tensile Strength, lb. per sq. in.	Elongation in 2 in., per cent	Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in. Ball, 100-kg. Load	Brinell Hardness	Scleroscope Hardness (Magnifier Hammer)
Electrolytic Copper, Gages 0.103 in. and thicker	34 200	50.3	−15.3	50.9	....
	41 400	19.2	44.3	85.7	....
	48 850	8.5	55.6	96.3	....
	53 100	7.0	59.4	100	....
	54 950	5.25	62.0	102	....
COPPER-ZINC ALLOYS:					
Alloy A, Gages 0.040 to 0.065 in	38 450	37.0	−4.0	53.4	....
	44 150	22.0	46.3	85.6	....
	46 850	10.0	56.4	91.3	....
	49 700	8.8	61.5	99.5	....
	61 650	4.0	72.9	117	....
	67 100	2.5	76.0	126	....
	69 900	3.5	78.0	136	....
	72 000	2.8	79.2	136	....
Alloy B, Gages 0.040 to 0.080 in	40 850	48.0	−1.0	....	....
	45 400	28.0	47.0	....	....
	51 800	16.0	64.6	....	....
	75 800	4.0	82.2	....	....
Alloy C, Gages 0.040 to 0.065 in	44 900	53.5	8.8	53.1	....
	50 550	35.0	56.1	87	....
	58 950	19.5	72.0	117	....
	71 850	8.5	83.7	136	....
	85 200	5.0	89.3	154.3	....
	91 350	4.4	91.0	164.7	....
Alloy D, Gage 0.065 in	46 200	68.0	8.3	....	....
	54 000	46.5	58.0	....	....
	60 500	32.5	70.0	....	....
	74 600	11.0	83.5	....	....
	83 800	7.5	88.1	....	....
	95 600	6.5	92.3	....	....
	98 600	6.5	93.5	....	....
Alloy E, Gage 0.040 in.:					
350° C. Anneal	56 000	....	45.1	....	....
	62 000	....	71.3	....	....
	70 000	....	83.0	....	....
	78 000	....	86.1	....	....
	85 000	....	89.2	....	....
	94 000	....	92.3	....	....
	98 000	....	93.7	....	....
	101 000	....	94.6	....	....
450° Anneal	52 000	....	29.6	....	....
	57 000	....	66.9	....	....
	64 000	....	77.5	....	....
	73 000	....	83.6	....	....
	80 000	....	87.9	....	....
	90 000	....	90.6	....	....
	95 000	....	93.1	....	....
	99 000	....	94.2	....	....
550° C. Anneal	48 000	....	15.2	....	....
	53 000	....	61.0	....	....
	60 000	....	74.1	....	....
	68 000	....	81.7	....	....
	76 000	....	86.0	....	....
	87 000	....	....	....	....
	92 000	....	92.0	....	....
	97 000	....	....	....	....
650° C. Anneal	45 000	....	2.0	....	....
	50 000	....	51.1	....	....
	57 000	....	70.9	....	....
	64 000	....	78.2	....	....
	72 000	....	83.9	....	....
	83 000	....	....	....	....
	89 000	....	90.5	....	....
	94 000	....	....	....	....
750° C. Anneal	43 000	....	−10.2	....	....
	47 000	....	45.0	....	....
	53 000	....	67.5	....	....
	61 000	....	76.5	....	....
	68 000	....	80.2	....	....
	80 000	....	85.7	....	....
	86 000	....	89.3	....	....
	92 000	....	91.9	....	....

TABLE II.—*Continued.*

	Tensile Strength, lb. per sq. in.	Elongation in 2 in., per cent	Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in. Ball, 100-kg. Load	Brinell Hardness	Scleroscope Hardness (Magnifier Hammer)
Alloy F, Gages 0.040 to 0.065 in.	47 050	57.5	4.0	53.4	....
	53 000	35.8	49.4	84.5	....
	53 450	34.5	52.0	85.6	....
	59 800	22.0	67.0	109	....
	73 350	8.0	82.7	140	....
	83 350	6.0	88.8	153	....
	90 050	5.3	91.2	158	....
Alloy G, Gages 0.040 to 0.091 in.	57 300	40.0	39.6	....	....
	66 100	33.5	74.4	107	....
	73 800	24.0	82.1	130	....
	80 000	17.3	86.0	136	....
	85 000	12.3	88.2	143	....
	90 100	8.5	90.7	155	....
	93 950	6.5	91.7	158	....
	96 600	6.3	92.2	159.5	....
	98 800	5.0	93.0	161.5	....
	101 500	4.75	93.2	166	....
	103 500	4.0	93.1	171	....
NICKEL SILVER ALLOYS:					
Alloy H, Gage 0.102 in.	54 500	....	41.0	69	....
	62 200	....	73.5	104	....
	70 400	....	81.5	119	....
	79 500	....	84.5	140	....
	82 800	....	87.0	150	....
Alloy I, Gages 0.052 to 0.162 in.	59 050	36.3	44.7	79.6	....
	64 400	28.5	72.7	89.0	....
	73 000	19.5	83.8	119	....
	80 100	6.0	87.0	126	....
	84 950	5.0	89.3	134.5	....
	89 400	4.5	91.0	136	....
	91 800	4.0	92.5	....	....
	94 300	3.5	94.2	....	....
	96 600	3.5	94.7	143	....
	98 500	3.0	95.8	150	....
	99 950	3.0	96.4	159	....
Alloy J, Gage 0.065 in.	66 100	45.5	51.0	....	....
	81 300	13.0	87.6	....	....
	93 400	5.0	92.8	....	....
	103 400	3.0	96.0	....	....
	108 600	3.5	97.9	....	....
	114 300	3.5	99.1	....	....
COPPER-TIN ALLOYS:					
Alloy K, Gages 0.042 to 0.103 in.	51 200	45.5	38.7	....	....
	64 500	19.5	80.0	....	....
	71 200	14.0	84.0	....	....
	87 200	6.3	91.8	....	....
	97 800	4.3	94.5	....	....
	105 100	3.0	95.4	....	....
Alloy L, Gages 0.040 to 0.081 in.	46 900	53.0	22.0	....	....
	61 200	21.8	75.5	....	....
	75 150	8.0	84.5	....	....
	85 650	6.0	91.2	....	....
	91 000	4.25	93.4	....	....
	97 850	3.25	94.9	....	....
	102 200	2.75	95.6	....	....
Alloy M, Gage 0.040 in.:					
470° C. Anneal	51 000	40.5	38.0	73	23.0
	57 500	26.5	70.6	108	40.5
	65 500	15.5	78.2	134	50.0
	75 500	10.0	85.9	154	56.0
	85 500	7.0	92.0	170	60.0
	97 000	4.0	93.3	188	65.0
	104 000	3.0	95.0	196	68.0
	109 000	2.7	98.3	201	70.0
570° C. Anneal	49 000	45.5	31.2	70	19.0
	55 000	30.5	67.6	103	37.0
	62 500	18.0	79.2	129	47.5
	72 500	11.5	82.8	149	54.0
	82 500	8.0	89.8	165	58.5
	93 500	4.5	92.6	183	64.0
	101 000	3.3	95.7	193	67.5
	107 000	2.8	97.0	198	70.0

TABLE II.—*Continued.*

	Tensile Strength, lb. per sq. in.	Elongation in 2 in., per cent	Rockwell Hardness, "B" Scale, $\frac{1}{16}$-in. Ball, 100-kg. Load	Brinell Hardness	Scleroscope Hardness (Magnifier Hammer)
670° C. Anneal	47 000	50.5	23.2	67	16.5
	52 500	34.5	61.2	98	34.5
	60 000	21.0	75.5	123	45.5
	69 500	13.0	82.7	143	52.0
	78 500	8.5	87.5	159	56.5
	90 000	5.0	92.2	178	61.5
	98 000	3.5	94.2	189	65.0
	104 000	2.8	95.5	195	67.0
770° C. Anneal	44 500	58.5	14.0	63	16.5
	50 000	37.0	54.0	93	32.5
	57 000	22.5	73.9	118	43.5
	66 000	13.5	79.8	138	50.0
	74 500	8.0	86.5	154	54.5
	86 000	4.5	91.0	174	60.0
	94 000	3.3	93.8	185	63.0
	100 000	2.8	95.1	192	65.0
870° C. Anneal	40 600	44.0	6.6	60	16.0
	46 000	29.0	60.0	88	31.0
	53 500	16.0	72.5	112	42.0
	62 500	8.5	79.0	132	49.0
	70 000	5.5	84.5	148	63.5
	81 000	3.5	88.7	170	59.0
	89 500	2.5	92.9	182	62.5
	96 000	2.5	94.9	189	65.0
Alloy N, Gage 0.040 in.	43 050	42.0	4.0	....	....
	44 250	48.5	7.5	....	....
	45 550	53.0	9.5	....	....
	46 550	50.8	12.8	....	....
	50 000	48.0	25.5	....	....
	52 550	45.0	36.0	....	....
	54 800	46.8	44.0	....	....
	56 200	42.8	50.5	....	....
	69 750	21.3	77.0	....	....
	75 300	19.3	83.5	....	....
	77 200	17.5	85.2	....	....
	80 700	15.8	86.0	....	....
	82 700	13.3	88.7	....	....
	82 950	10.0	89.3	....	....
	83 300	5.8	89.3	....	....
	83 900	5.8	89.0	....	....
Alloy O, Gages 0.046 to 0.131 in.	48 500	....	26.2	....	....
	55 550	42.5	69.8	....	....
	65 800	23.0	81.5	....	....
	73 450	14.5	85.7	....	....
	80 850	10.3	89.4	....	....
	89 000	6.0	92.6	....	....
	91 900	5.5	93.6	....	....
	96 600	4.8	94.6	....	....
	100 500	3.0	96.0	....	....
	101 500	2.5	96.7	....	....
Alloy P, Gages 0.041 to 0.064 in.	57 750	73.5	45.5	....	....
	71 200	36.0	83.0	....	....
	80 400	33.0	90.2	....	....
	86 900	17.0	92.2	....	....
	95 700	10.0	95.0	....	....
	99 200	8.5	96.0	....	....
	106 500	5.5	97.8	....	....
	116 800	4.0	101.0	....	....
Alloy Q, Gages 0.042 to 0.107 in.	65 850	62.5	52.0	....	....
	81 300	36.8	87.4	....	....
	103 750	15.5	97.6	....	....
	115 150	8.3	100.0	....	....
	122 850	6.3	100.8	....	....

Schwarz states that a general linear relationship between tensile strength and hardness, for alloys like brass and bronze, cannot be derived. The hardness relationships of each alloy will have to be specifically determined.

In order to throw further light upon the hardness relationships of some of the principal copper alloys, those of the copper-zinc system and those of the copper-tin system (alpha phase) have been studied individually as well as in groups. Three nickel silvers were selected to represent a portion of the ternary system. The evidence of tension tests, and Brinell, Rockwell and scleroscope hardness values on annealing and rolling series has been assembled in tables of data and plotted in eleven charts (Figs. 1 to 11) under which, rather than in the text, is the explanation of each figure. No attempt has been made to cover the whole field but rather to show typical examples of the several relationships. A further complexity due to the two last rolling and annealing operations has been demonstrated but is not included in the present paper.

The intent of the figures is to show:

1. That the hardness - tensile strength relationship for a given copper alloy is not a simple one, but is as complex as the previous history of the material.
2. The hardness - tensile strength relationships for the several copper alloys in a binary system (alpha phase) change progressively from that of pure copper to that of the alloy of lowest copper content.
3. The hardness - tensile strength relationships of ternary alloy systems (alpha phase) change in a manner similar to that of binary systems. An example is given of a series of three copper-zinc-nickel alloys, all containing 18 per cent of nickel, but with a copper content ranging from 71.31 to 55.59 per cent.
4. The hardness relationships themselves (Brinell to Rockwell to scleroscope, etc.) are determined by the governing conditions of composition and previous history of the material involved.
5. The tensile strength - elongation, and hardness - elongation relationships are likewise determined by the composition and previous history of the material.
6. The tensile strength - hardness relationship of an annealing series of an alloy is a single line. Any of the anneals may be selected as the beginning of a distinct rolling series, but the tensile strength - hardness relationships of the latter, while similar to each other in form, deviate from and are quite unlike the annealing series curves (see Figs. 2 and 7).

The practical importance of the data here presented may not be immediately apparent. A specification always recognizes the difficulty of casting metal to an exact formula; it also makes allowance for gage tolerances in rolling, and the variations in reduction that may result. There is a real danger that a specification having, for instance, tensile strength limits, may be redesigned to include hardness test limits chosen from a composite conversion curve.

It is the purpose of this paper to point out that such a practice is wrong, and that the hardness relationships for bars even of the *same* chemical composition may vary appreciably. The variation in the hardness relationships of bars of *different* chemical composition is so great that every effort should be made to secure the exact conversion curve or curves for each alloy under consideration. The arrangement of the data in the charts enables one furthermore to select a desired combination of qualities in an alloy, or points the way toward the solution of similar problems with other alloys.

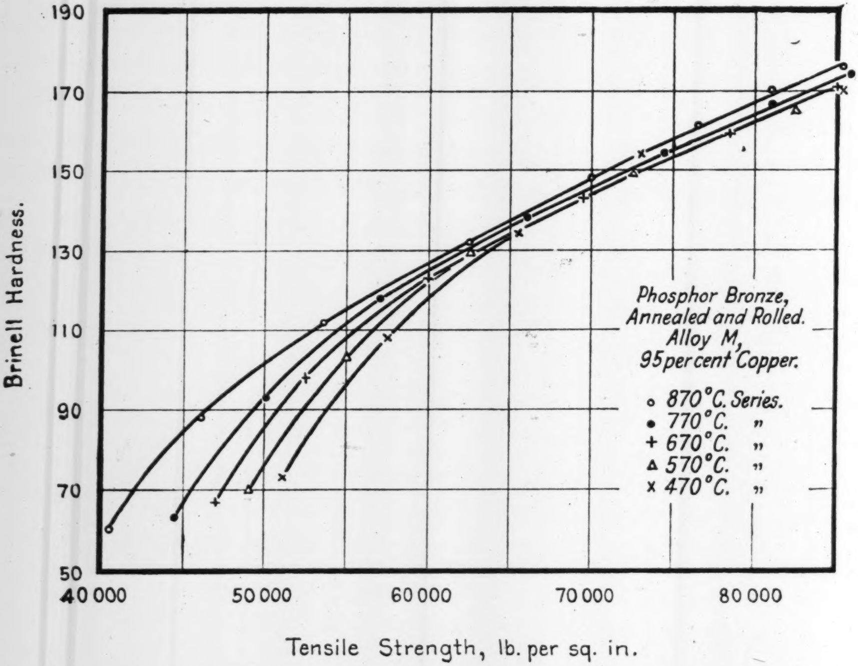

FIG. 1.—Relation of Brinell Hardness to Tensile Strength for Five Rolling Series of a Single Copper-Tin Alloy, Alloy M.

A bar was annealed at each of the temperatures 470° C., 570° C., 670° C., 770° C., and 870° C., and then rolled. The Brinell-hardness - tensile-strength relationships vary directly, because of the effect of the anneals previous to rolling.

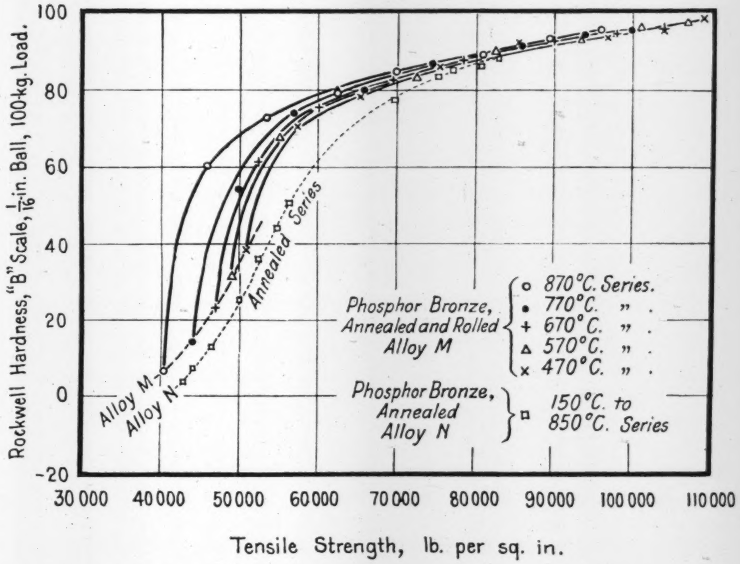

FIG. 2.—Relation of Rockwell Hardness to Tensile Strength for Rolling Series as in Fig. 1, Copper-Tin Alloy M.

The Rockwell-hardness - tensile-strength relationships very directly in the anneals given previous to rolling. The single hardness - tensile-strength relationship curve of the annealed specimens of alloy M is compared with that of an annealing series on a similar alloy, N. Contrast the slope of this typical annealing curve with those of the typical rolling series just above (compare Fig. 7).

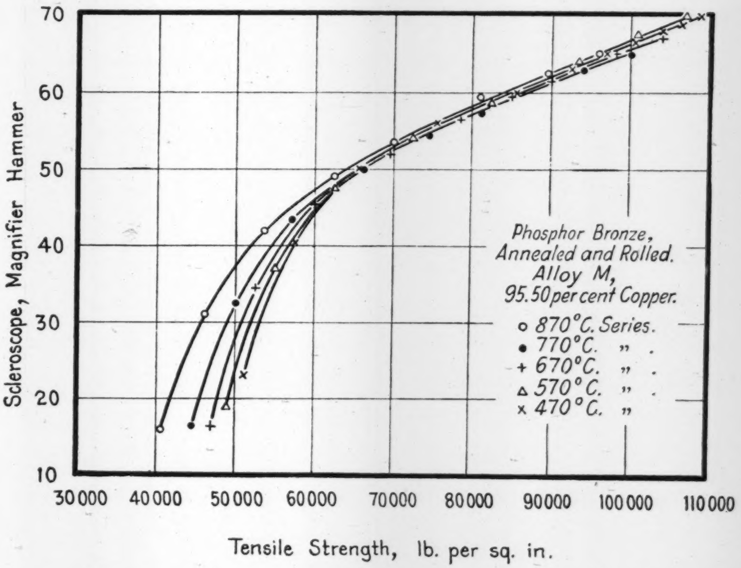

FIG. 3.—Relation of Scleroscope Hardness to Tensile Strength for Rolling Series as in Fig. 1, Copper-Tin Alloy M.

The scleroscope - tensile-strength relationship curves are similar to those shown for Brinell and Rockwell hardness, Figs. 1 and 2.

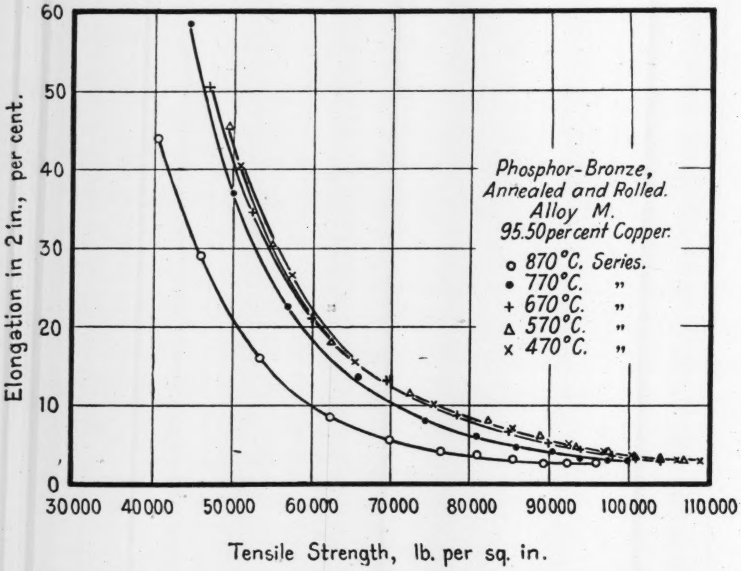

FIG. 4.—Relation of Tensile Strength to Percentage of Elongation for Rolling Series as in Fig. 1, Copper-Tin Alloy M.

The tensile-strength - elongation relationships also vary because of the effect of the anneals previous to rolling.

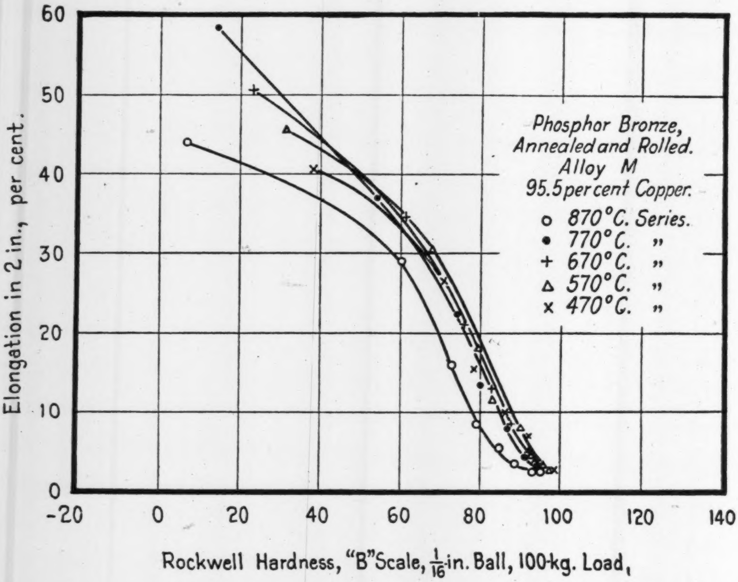

FIG. 5.—Relation of Rockwell Hardness to Percentage of Elongation for Rolling Series as in Fig. 1, Copper-Tin Alloy M.

The Rockwell-hardness - elongation relationships differ markedly in relative position from those of tensile-strength - elongation shown in Fig. 4.

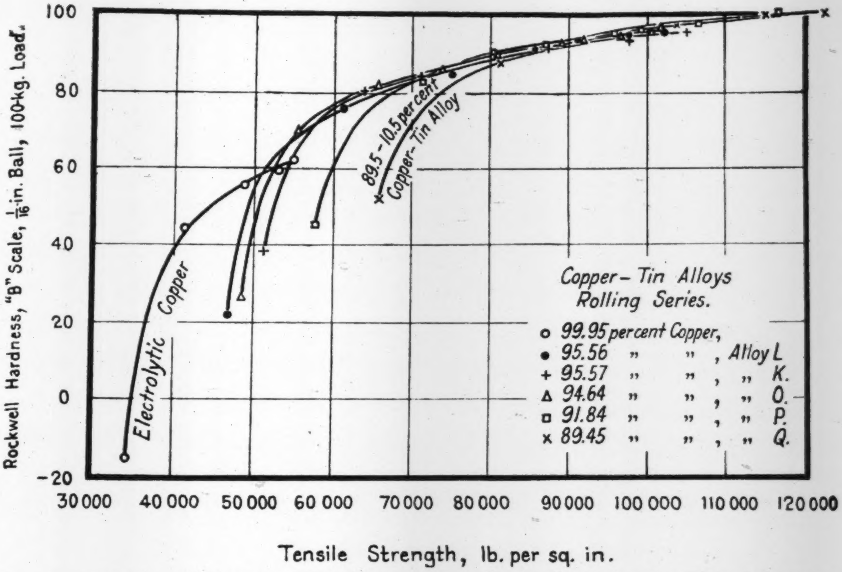

FIG. 6.—Relation of Rockwell Hardness to Tensile Strength for the Binary System, Copper-Tin Through the Alpha Phase.

The relationships vary directly with the percentage of copper (compare Fig. 7). A curve for commercial electrolytic copper is shown for comparison.

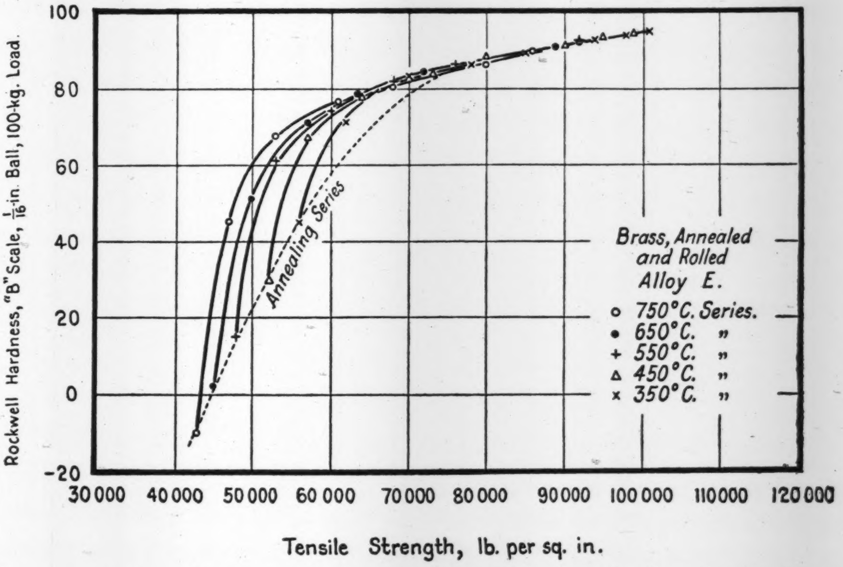

FIG. 7.—Relation of Rockwell Hardness to Tensile Strength for Five Rolling Series on a Single Copper-Zinc Alloy E.

A bar was annealed at each of the temperatures 350° C., 450° C., 550° C., 650° C., and 750° C., and then rolled. The Rockwell-hardness - tensile-strength relationships vary directly because of the effect of the anneals previous to rolling. The dotted line shows the single line relationship of the annealing series of this alloy (compare Fig. 2.).

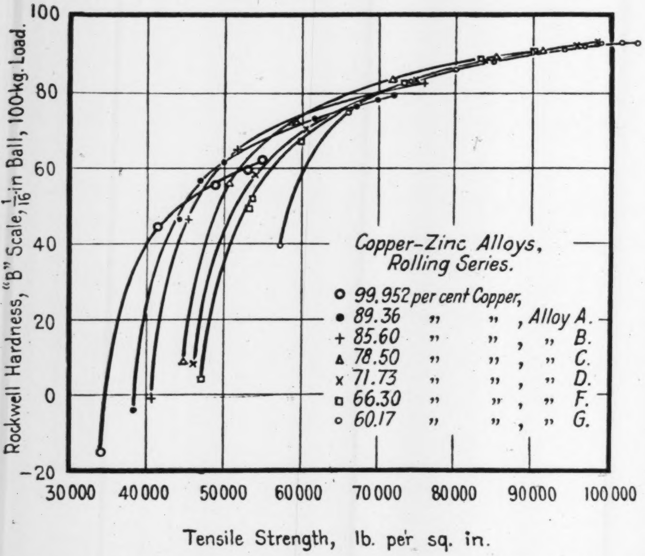

FIG. 8.—Relation of Rockwell Hardness to Tensile Strength for the Copper-Zinc System in the Alpha Phase.

The relationships vary directly with the percentage of copper (compare Fig. 6). The plot does not show the greater intricacy introduced by variations in the annealing and rolling practice. A curve for commercial electrolytic copper is shown for comparison.

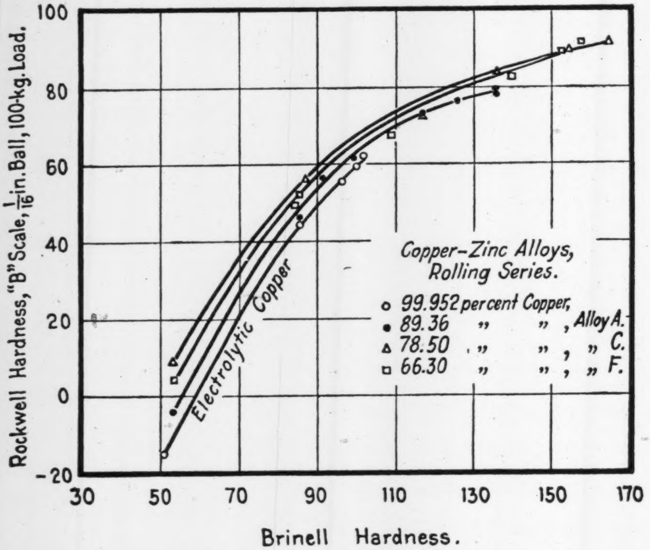

FIG. 9.—Relation of Rockwell Hardness to Brinell Hardness for Three Copper-Zinc Alloys.

The relationships indicate that maximum conversion values lie between 70 and 80 per cent copper (compare Fig. 10). A curve for commercial electrolytic copper is shown for comparison.

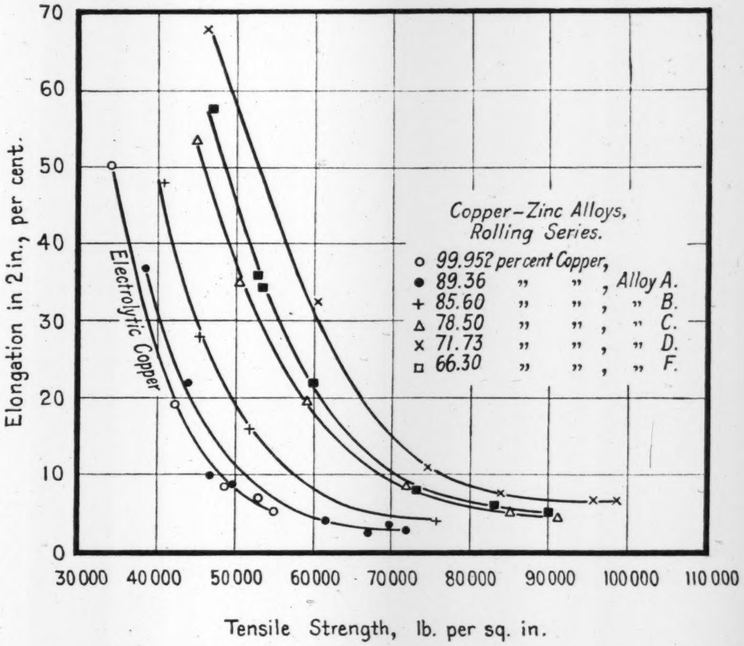

FIG. 10.—Relation of Tensile Strength to Percentage of Elongation for the Copper-Zinc System.

The maximum values for both tensile strength and elongation are in the vicinity of 72 per cent copper. A curve for commercial electrolytic copper is shown for comparison. See W. H. Bassett and C. H. Davis, "Physical Characteristics of Commercial Copper-Zinc Alloys," *Proceedings*, Inst. of Metals Division, Am. Inst. Mining and Metallurgical Engrs., p. 55 (1928).

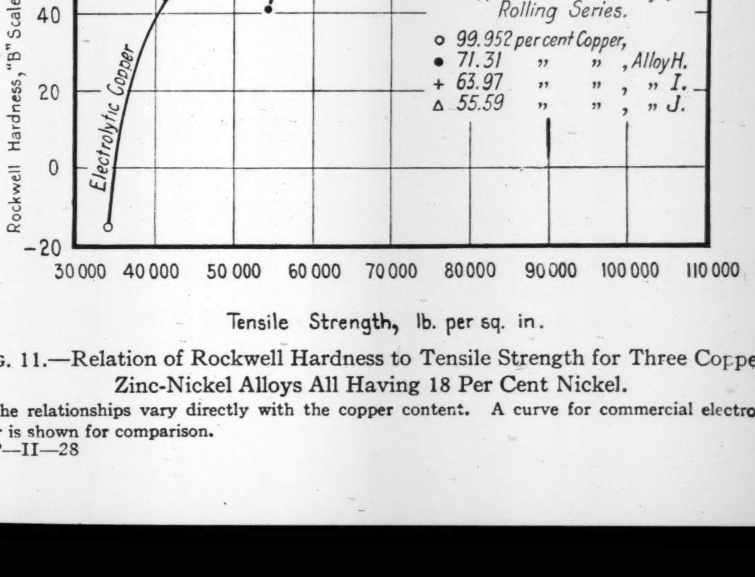

FIG. 11.—Relation of Rockwell Hardness to Tensile Strength for Three Copper-Zinc-Nickel Alloys All Having 18 Per Cent Nickel.

The relationships vary directly with the copper content. A curve for commercial electrolytic copper is shown for comparison.

DISCUSSION

MR. I. H. COWDREY[1] (*presented on written form*).—An examination of the paper by Messrs. Davis and Munson leads the writer to call attention to several items that seem to have been completely overlooked by the authors. It is felt that their attempt to establish the necessity for a multiplicity of curves to show the Rockwell-Brinell relation is not justified when certain previously established facts are taken into consideration. Since Fig. 9 of the paper is the only graph devoted to this relation, this discussion will be confined to the results of tests on alloys A, C and F.

Since no statement appears to the contrary it is assumed that the Brinell tests have been made with the standard light pressure of 500 kg., and the standard ball of 10-mm. diameter. The specimens listed appear to be no more than 0.065 in. in thickness in any instance. In a paper presented before the American Society for Steel Treating in September, 1925, Mr. S. N. Petrenko of the Bureau of Standards recommends that "no specimen should be used thinner than 0.35 in." The use of specimens thinner than that value will result in a restriction of the metallic flow and the harder supporting plate will very materially influence the results. Brinell values obtained with thin specimens will be very appreciably raised. The softer the specimen the greater will be this effect. This item will cause the curves of Fig. 9 to fall to the right of their true location and the shift will be greatest in the lower ranges of the curve.

The first paper to be published establishing the single line relation between the Rockwell and Brinell scales was that presented by the writer before the American Society for Steel Treating in September, 1924. In that paper it was demonstrated that non-ferrous materials may show wide variations in Rockwell hardness even over a very small area. As an extreme case that paper showed results varying from $(R_{16}) - 19$ to $(R_{16}) - 48$ over less than 1 sq. in. area with a certain piece of brass in which the grain boundaries were distinctly visible with the unaided eye. While this is an extreme example, variations of several points may be expected in the case of non-ferrous materials as was clearly demonstrated by a study of the original data from which the paper in question was prepared.

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[1] Associate Professor of Testing Materials, Massachusetts Institute of Technology, Cambridge, Mass.

In a paper presented before the American Society for Steel Treating in October, 1923, Mr. A. L. Davis shows that with a "single homogeneous sample of brass" both the Brinell and Rockwell values showed a variation of plus or minus 3 points from the mean.

Using as a basis the expectable variations quoted by A. L. Davis, a new plot has been constructed by the writer as follows. Data from the paper by Davis and Munson have been plotted in the accompanying Fig. 1. About each point so determined there has been circumscribed a square which represents the possible plus and minus variations that may be expected with material which would be considered "homogeneous." Now the point plotted might be expected to fall anywhere inside this area if other observations were made. The data for alloy C have been omitted since Fig. 9 shows this curve to lie between those for alloys A and F.

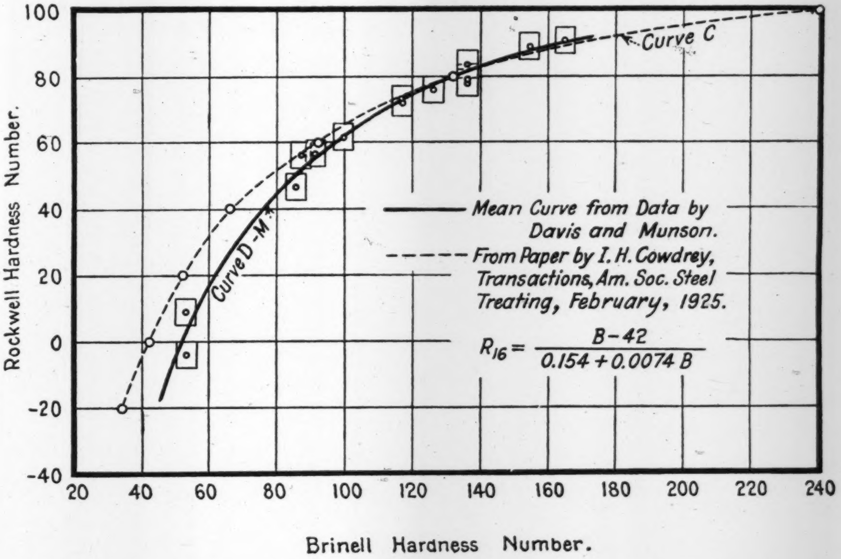

FIG. 1.—Showing Relation of Rockwell Hardness to Brinell Hardness of Non-Ferrous Metals.

A representative curve has been drawn which it is thought averages the plotted data. It will be noted that this "mean curve," marked "*D–M*," passes through some part of every one of the circumscribed areas. This indicates that every sample studied might have yielded data giving a point exactly on this mean line.

A dotted line has been drawn reproducing the original graph deduced by the writer in the 1924 paper to which reference has already been made. It will be noted that this line is coincident with the "mean D–M line" in the upper ranges but shows appreciable deviation in the lower portion. This condition is entirely in accordance with what might be expected from the thickness factor previously outlined in this discussion.

As a result of the foregoing analysis and certain research studies conducted under his supervision, the writer is convinced that a "single line conversion chart" is sufficiently reliable for determining the Rockwell-Brinell relation. Variations arising from its use are less than the variations which may be expected in a series of observations made on the same piece of metal. When specimens are used as thin as those used by Messrs. Davis and Munson, it is apparent that an error appears of such magnitude that conclusions drawn from the resulting data must be interpreted with great caution.

MR. R. L. TEMPLIN.[1]—The authors of this paper have neglected to indicate the kind, size and shape of the test specimens used in obtaining the tensile properties given. It has been pointed out very definitely at previous meetings of the Society that the elongation values discussed by the authors are dependent to a very large extent on the factors just mentioned and some of the relationships shown might be better explained if this fact were taken into account. Would the authors give us more information on the points just indicated?

MR. C. H. DAVIS[2] (*authors' closure*).—The refinements of Rockwell hardness testing have been largely developed since the publication of Mr. Cowdrey's paper in 1925. In his paper Cowdrey did not give any of the gages of the material, any tensile strength values, any analyses, and the plotted points were not identified with the particular kind of material tested.

The authors feel quite justified in establishing a multiplicity of curves to show hardness relationships in view of the demand for close specifications for copper alloy material. The standard Brinell test (10-mm. ball, 500-kg. load) is accurate on yellow brass to a thickness of about 0.080 in. on annealed, and to 0.050 in. on hard or rolled material. This was demonstrated by Fig. 6 of a paper presented before the Society by one of the present authors in 1917.[3] Figure 8 of that paper likewise shows that the standard Brinell test is fairly accurate on brass rolling series on gages down to about 0.040 in. The statement of Mr. Petrenko in U. S. Bureau of Standards *Tech-*

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[1] Chief Engineer of Tests, Aluminum Co. of America, New Kensington, Pa.

[2] Assistant Metallurgist, The American Brass Co., Waterbury, Conn.

[3] C. H. Davis, "Testing of Sheet Brass," *Proceedings*, Am. Soc. Testing Mats., Vol. XVII, Part II, p. 164, (1917).

nologic Paper No. 334 that the Brinell test is not accurate for brasses thinner than 0.350 in. would throw the Brinell test out almost entirely in the brass industry.

The error introduced into the paper now under discussion by Brinell test of specimens 0.040 in. to 0.065 in. would be of the order of 6 per cent on the softer, and negligible on the harder specimens; even so, the entire set of curves in Fig. 9 of the paper would move together to the left. Consequently the difference in the relationship pointed out by the authors still stands.

We do not wish to consider a premise allowing plus or minus 3 Rockwell points on a specimen. The refinement introduced into Rockwell testing and the demands of consumers of material allow only plus or minus 1½ points on the softest brasses and only a fraction of a point on the hardest copper alloys. In this connection reference may be made to the paper presented at this annual meeting by Messrs. J. R. Townsend, W. A. Straw and C. H. Davis on "Physical Properties and Methods of Test for Some Sheet Non-Ferrous Metals."[1]

In reply to Mr. Templin, we would state that the types of specimens used were those described in a paper by Messrs. Van Deusen, Shaw, and Davis, presented before this Society in 1927,[2] and in the paper on "Testing of Sheet Brass" presented in 1917. The percentage elongation in 2 in. was determined in all cases. The gages of the specimens are given either precisely or in the range of a rolling series. All plotted comparisons of elongation data were from specimens of approximately the same gage.

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[1] See p. 381.

[2] H. N. Van Deusen, L. I. Shaw, and C. H. Davis, "Physical Properties and Methods of Test for Sheet Brass," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 173 (1927).

AN X-RAY STUDY OF COPPER WHICH SHOWED DIRECTIONAL PROPERTIES ON CUPPING

By Arthur Phillips[1] and Gerald Edmunds[2]

Synopsis

This paper reviews briefly the important literature dealing with the orientation characteristics of copper in the hard-rolled and annealed conditions. An X-ray study, by the monochromatic pinhole method, of annealed copper which showed directional properties on cupping is described. Fiber photographs of rolled and annealed metal are given, together with analyses of the structures involved. It is suggested that pronounced directional properties may be eliminated by limiting the degree of cold working and avoiding excessively high annealing temperatures.

It is probable that the most fascinating and, at the same time, most elusive problem of general interest to present-day metallurgists has to do with the related studies of the mechanism of plastic deformation, recrystallization, and grain growth. Until about ten years ago, the results of the numerous investigations on cold-worked and annealed metals were expressed in terms of the changes in grain characteristics and mechanical properties produced by varying the degree of cold working and the temperature and time of annealing. During the past few years, however, X-ray studies have contributed information of a fundamental and quantitative character regarding the changes within the crystal lattice when metals are cold worked or heat treated.

Orientation Characteristics

It has been fairly well established that the cold working of polycrystallin metal produces fragmentation of the grains, and that an original grain, after deforming, is composed of a number of fragments differing somewhat in orientation.[3] The degree of fragmentation and the orientation of the fragmented particles are undoubtedly determined primarily by the magnitude and nature of the stresses applied.

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[1] Associate Professor of Metallurgy, Yale University, New Haven, Conn.

[2] Graduate Student, Department of Mining and Metallurgy, Yale University, New Haven, Conn.

[3] E. C. Bain and Zay Jeffries, "Mixed Orientations Developed in Crystals of Ductile Metals by Plastic Deformation," *Chemical and Metallurgical Engineering*, Vol. 25, p. 775 (1921).

In fact, extremely heavy reductions lead to pronounced directional properties as a result of the formation of a preferred orientation with reference to the axis of extension. This condition is commonly known as "fiber" and is not to be confused with the type of fiber caused by the elongation of slag inclusions and other secondary or incidental constituents.

According to the literature, the preferred orientations developed by the cold working of metals having the same lattice characteristics are not the same. Moreover, the type of fiber in a given metal is dependent on the nature of the applied stresses as has been demonstrated by the studies of metals deformed by the two common methods of working, namely, drawing and rolling. In other words, the planes in the surface and the grain orientation with respect to the direction of extension are not usually the same in drawn and rolled metals.

Orientation Characteristics of Cold-Rolled Copper:

When copper is plastically deformed by cold rolling, extension takes place by slip upon the octahedral (111) planes in a 110 direction. According to Mark and Weissenberg,[1] Glocker,[2] and others, in severely-rolled copper the grain fragments are oriented with a 112 direction parallel to the rolling direction and a 110 plane in the rolled surface. This orientation has been rather generally assigned to rolled metals of the face-centered cubic system. It is pertinent to note in this connection that practically all of the X-ray studies reported on rolled metals have been restricted to severely worked sheet in which the reductions have ranged from approximately 95 to 98 per cent. There is abundant reason for believing that the more moderate reductions associated with rolling mill practice lead to conditions differing, at least quantitatively, from the results described in drastically worked metals.

Tammann and Meyer,[3] by their method based on maximum reflection (*maximale schimmer*) have analyzed the orientation changes as a function of the degree of rolling. They reported that as the reduction of randomly oriented copper progresses, up to about 30 per cent, the number of 111 faces in the surface increases, with an almost corresponding decrease in the number of 100 faces. Apparently the reduction necessary to produce the maximum number of 111 faces in the surface depends somewhat on the orientation characteristics of the annealed material; the maximum occurs in all cases, however, at reductions of 30 to 50 per cent. As the degree of rolling increases there is a reduction in the number of 111 and 100 planes with a progressive increase in the number of 110 planes in the surface. After a 70-per-cent reduction their analysis shows that 6 per cent of the 100, 20 per cent of the 111, and 73 per cent of the 110 planes lie in the sheet surface. After somewhat higher reductions the rolled surface shows only 110 planes with a 112 direction in the rolling axis. Tammann and Meyer's results on hard-rolled copper are in harmony with the X-ray studies previously mentioned.

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[1] H. Mark and K. Weissenberg, "Röntgenographic Determination of the Structure of Rolled Metal Foils," *Zeitschrift Physik*, Vol. 14, p. 328 (1923); also Vol. 16, p. 314 (1923).

[2] R. Glocker, "Materialprüfung mit Röntgenstrahlen," Julius Springer (1927).

[3] G. Tammann and H. H. Meyer, "Über die Kristallitenorientierung in Abhängigkeit vom Walzgrade beim Kupfer," *Zeitschrift Metallkunde*, Vol. 19, p. 82 (1927).

TABLE I.—RESULTS OF DETERMINATIONS BY THE MAXIMUM REFLECTION METHOD, OF THE FACES PRESENT IN SURFACE OF ANNEALED COPPER.

Temperature of Annealing, deg. Cent. (Time 2 hours)	Faces Present in Surface of Copper					
	Rolled 87-per-cent Reduction			Rolled 33-per-cent Reduction		
	(111)	(011)	(001)	(111)	(011)	(001)
600	76	24	0	..	..	..
700	68	30	2	64	26	0
750	56	38	6	70	30	0
800	36	40	24	38	46	16
900	28	48	24	..	..	..
1000	22	12	66	..	..	..
1050	0	3	97	0	5	95

Orientation Characteristics of Annealed Copper:

According to Glocker[1] and Göler and Sachs,[2] copper after recrystallization shows a preferred orientation (at all temperatures from about 200° C. to the melting point) characterized by a cube (100) plane in the surface and a cube (100) edge in the rolling direction. It will be suggested elsewhere, as a result of the authors' work, that this extreme condition of preferredness is not characteristic of copper given moderate reductions followed by heating within the usual annealing range.

Tammann and Meyer,[3] have determined by their maximum reflection method the faces present in the surface of annealed copper previously reduced 33 and 87 per cent. Some of their results are assembled in Table I.

Tammann has calculated that random orientation is attained when the crystal planes in the surface are present in the following ratio:

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[1] R. Glocker, "Materialprüfung mit Röntgenstrahlen," Julius Springer (1927).

[2] V. Göler and G. Sachs, "Walz- und Recrystallisationstextur Regulär-Flächenzentrienter Metall II," *Zeitschrift Physik*, Vol. 41, p. 889 (1927).

[3] G. Tammann and H. H. Meyer, "Die Änderung der Kristallitenorientierung bei Rekristallisation von Kupfer," *Zeitschrift Metallkunde*, Vol. 18, p. 176 (1926).

111, 38 per cent; 110, 40 per cent; and 100, 22 per cent. From Tamman and Myer's results it would appear that this condition is approximated by annealing at about 800° C. They did not find the cube position described by Glocker and others at temperatures below approximately 1050° C.

Significance of Fiber in Metal Technology:

In many metal manufacturing operations the anisotropy resulting from preferred orientations is frequently manifested by the undesirable formation of "ears" or "tips" formed on drawn objects such as cups. The ears are usually symmetrically located with respect to the direction of rolling of the strip from which they are drawn. The formation of ears is particularly common in the drawing of copper, low brass, aluminum, iron and nickel silver.

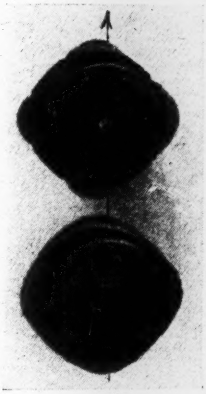

FIG. 1.—Copper Cups Showing Formation of Ears.

Kaiser,[1] one of the few writers on this subject, states that tipping is due to faulty annealing and rolling schedules and that the location of the ears is determined primarily by the temperature of the next to the last anneal.

Specific Example of Tipping in Copper Cups.—An excellent example of tipping in electrolytic copper (copper 99.96 per cent) was recently brought to the attention of the authors. The ears, four in number, were distributed with one pair directly on the rolling axis and the other pair at 90 deg. to the axis. Typical cups are shown in Fig. 1. The cups are $1\frac{1}{8}$ in. in diameter, $\frac{5}{8}$ in. in depth, and 0.013 in. in thickness.

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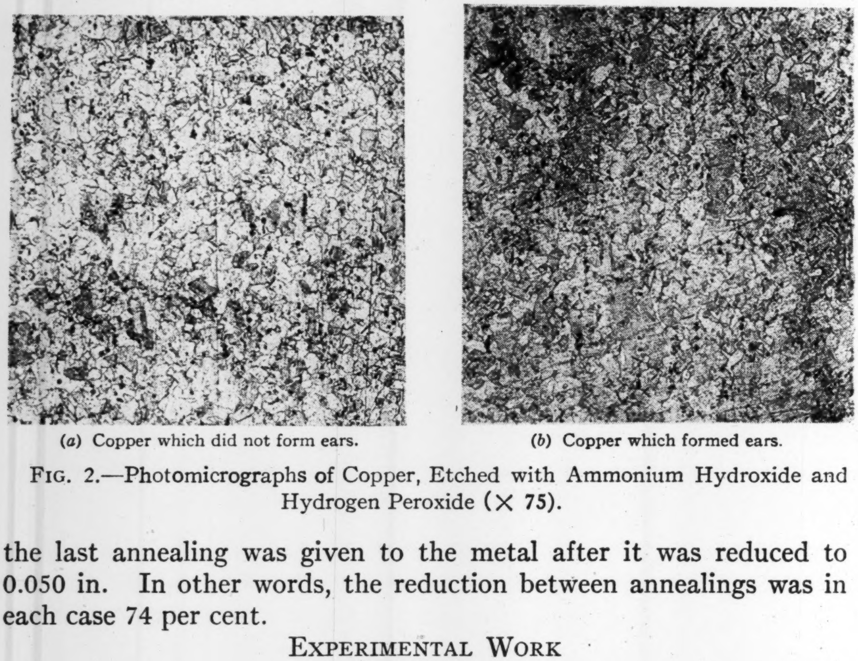
[1] K. Kaiser, "Die Entstehung von unebenen Rändern an Hohlkörpen aus Kupferblech," *Zeitschrift Metallkunde*, Vol. 19, p. 435 (1927).

Approximately 50,000 lb. of this copper rolled according to the following mill schedule drew without appreciable tipping:

Hot-rolled to 0.192 in.,
Cold-rolled in 7 passes to 0.043 in. (reduction 77.5 per cent),
Annealed at 550° C.,
Cold-rolled in 4 passes to 0.013 in. (reduction 70.0 per cent),
Annealed at 550° C.

The copper which formed ears was rolled according to a modified rolling program and from the best information available the next to the last annealing was given to the metal after it was reduced to 0.050 in. In other words, the reduction between annealings was in each case 74 per cent.



(*a*) Copper which did not form ears. (*b*) Copper which formed ears.

FIG. 2.—Photomicrographs of Copper, Etched with Ammonium Hydroxide and Hydrogen Peroxide (× 75).

EXPERIMENTAL WORK

Micrographs of the satisfactory and unsatisfactory sheets are reproduced in Figs. 2 (*a*) and (*b*), respectively. The copper which showed no directional tendencies on drawing shows approximately equi-axed grains consistent in size with the annealing reported. In the case of the unsatisfactory copper, relatively large grains, elongated in the direction of rolling, are found together with normal equi-axed grains.

It seemed desirable to analyze the structural conditions of the rolled (but not annealed) copper by X-ray diffraction according to the monochromatic pinhole method, which is particularly suited to this type of investigation. By this method, the diffraction pattern of a polycrystallin aggregate of random orientation consists essen-

PLATE I.
PROC. AM. SOC. TESTING MATS.
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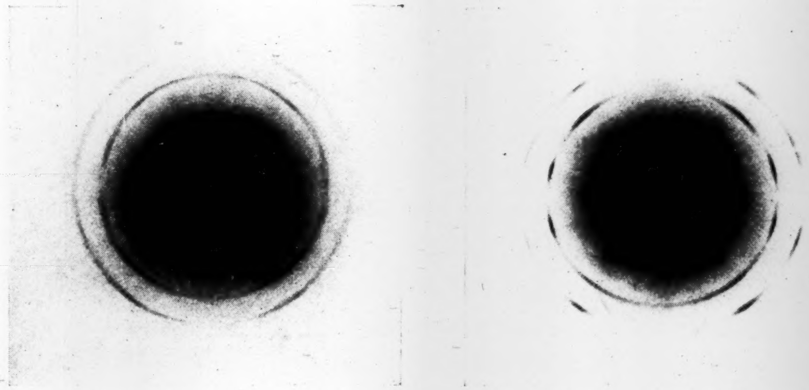
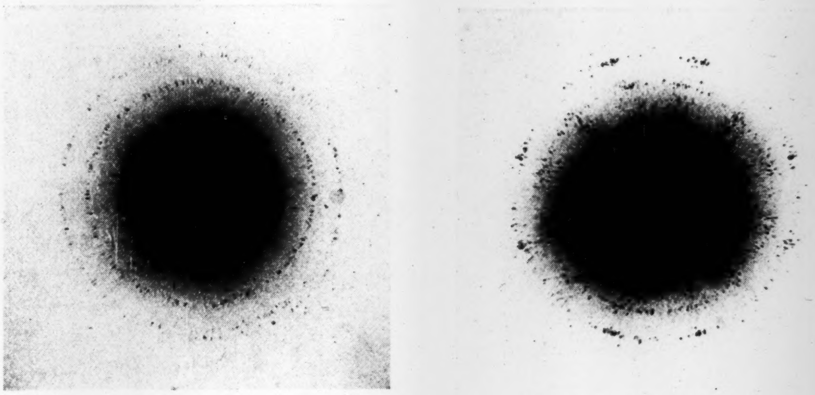
PHILLIPS AND EDMUNDS:
X-RAY STUDIES OF COPPER.



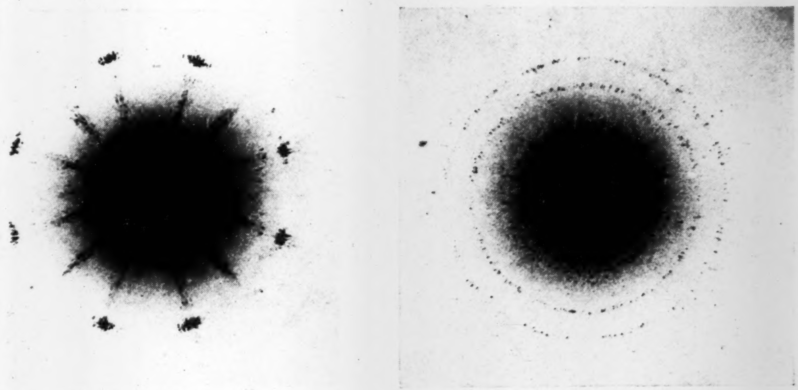
(*a*) Copper cold rolled 70 per cent.

(*b*) Copper cold rolled 99 per cent.



(*c*) Copper which did not form ears.

(*d*) Copper which did form ears.



(*e*) Copper cold rolled 99 per cent, then annealed at 525° C. (15 min.).

(*f*) Copper cold rolled 70 per cent, then annealed at 525° C. (15 min.).

FIG. 3.—Monochromatic Pinhole Diffraction Pattern of Copper.

tially of a series of concentric rings of uniform intensity whose diameters are determined by the size and type of the lattice, the X-ray wave length and the distance between the material and the photographic plate. For copper which is face-centered cubic with a cube edge of 3.60×10^{-8} cm. the rings, beginning with the smallest, are due to the diffraction of the X-ray beam from the following planes: 111, 100, 220, 311, 222, etc.

If, however, there is a common orientation for a considerable number of crystals or crystallin fragments some segments of the circles will receive an increased amount of radiation while other parts of the rings receive less. In extreme cases of preferredness the rings are not continuous but consist of broken parts of a circle. In order to state in precise terms the fiber characteristics it is necessary to know the direction in the lattice which lies parallel to the axis of extension and to identify the crystal plane in the surface of the sheet.

In the present investigation a standard multiple diffraction apparatus (General Electric Co.) containing a Coolidge molybdenum target tube operating at 30,000 volts and 25 milli-amperes was used. The specimens, after being reduced in thickness to about 0.0035 in. by etching with concentrated nitric acid, were mounted over the 0.02-in. square pinholes, and exposed for 45 hours to the K_a radiation directed normal to the rolling surface. Figures 3 (*a*) and (*b*) (Plate I) are the patterns obtained on copper reduced 70 and 99 per cent by rolling.

In order to learn the direction in the lattice parallel to the direction of rolling, angular measurements were made on the film between lines connecting opposite corresponding points and a line parallel to the rolling direction. These values were obtained for each of the interference circles. The cosine of this angle multiplied by the cosine of the diffraction angle[1] is equal to the cosine of the angle between the reflecting plane and the rolling direction.[2] By a direct comparison of the angles determined in this way with a table of angles between the important crystallographic directions, it was found that the direction of rolling coincides with the $\bar{3}53$ direction of the lattice. An inspection of the Laué spots at the center of the film indicated that a 101 plane, which is the only crystallographically important plane whose normal is at 90 deg. to the $\bar{3}53$ direction, lies in the sheet surface. The $\bar{1}21$ direction, which has been found by other investigators, deviates from the $\bar{3}53$ by only 5 deg.

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[1] The diffraction angle θ for cubic crystals is given by the equation, $\theta = \sin^{-1} \frac{\lambda(h^2 + k^2 + l^2)^{\frac{1}{2}}}{2a_0}$ where λ is the wave length; h, k and l are the crystallographic indices of the reflection plane; and a_0 is the length of the unit edge.

[2] R. Glocker, "Materialprüfung mit Röntgenstrahlen," Julius Springer (1927).

In order to verify this interpretation, a stereographic projection (Fig. 4) of the important planes of the face-centered cubic lattice was prepared with a $\bar{1}01$ projection at the center, and the $\bar{3}53$ at the uppermost point of the fundamental circle. The stereographic projections of the Debye-Scherrer circles (the origins of which are indicated by

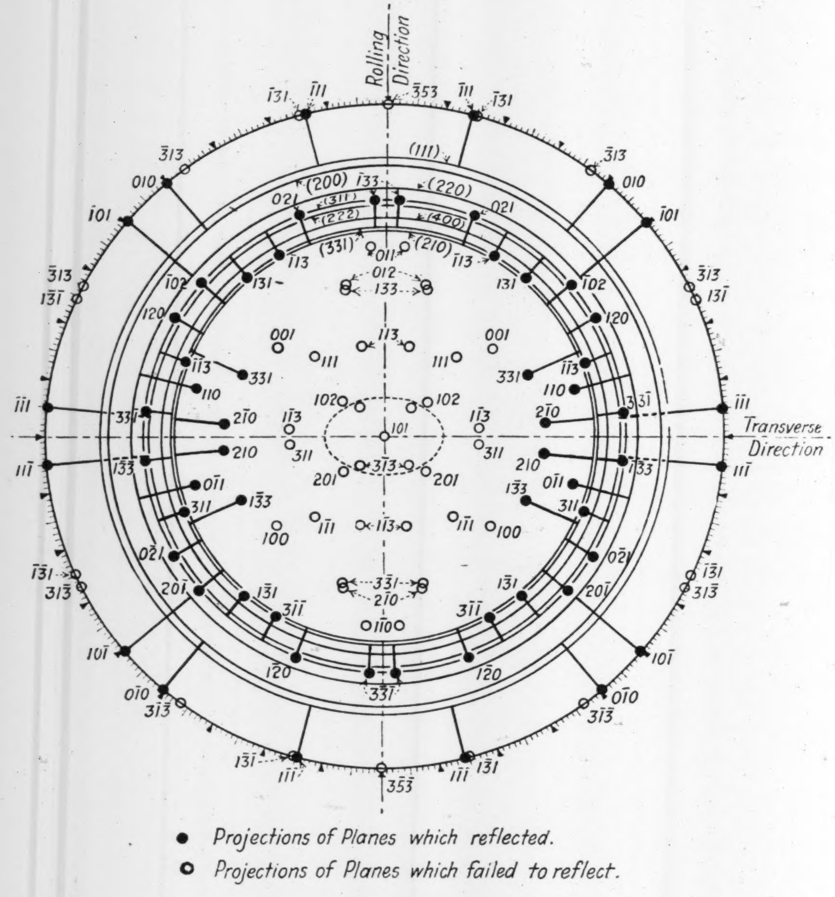
Rolling Direction
Transverse Direction
353
(111)
(200)
(220)
(311)
(222)
(400)
(331)
(210)
101
● Projections of Planes which reflected.
○ Projections of Planes which failed to reflect.

FIG. 4.—Stereographic Projection.

indices in parentheses) are included in the diagram. The projections of the planes which failed to reflect are indicated by hollow circles, and the others by full circles; the latter are connected by radial lines to the projections of the diffraction circles on which reflection occurred. It will be noted that there are two sets of spots which have been assigned the same indices, indicating the two positions of the lattice considered as mirror images across a plane parallel or perpendicular to the rolling direction and perpendicular to the rolling surface. An excellent agreement was found to exist between the photographs (Figs. 3 (*a*) and (*b*)) and the stereographic projection.

It is evident from the photographs and the projection that the inclination of the 101 plane to the rolling surface is greater about the rolling axis than about the transverse direction. This is shown by the elliptical curve which indicates that the relative variation of 101 planes from the mean position (Fig. 4) is approximately 20 deg. about the rolling direction as compared to 14 deg. about the transverse axis. The deviation about the normal direction is less than 4 deg. The presence of reflections of the same planes in the material reduced 70 per cent (Fig. 3 (*a*)) suggests that the inclination of the 101 planes to the surface is of the same order as in the harder rolled material, but that the deviation about the normal to the rolling surface is considerably greater.

X-ray Examination of the Annealed Material:

Figures 3 (*c*) and (*d*) are pinhole diffraction patterns of the satisfactory and defective copper. With the larger grain size due to annealing, many crystal positions are absent and each individual interference point becomes intense on account of the simultaneous action of many parallel planes. This condition produces interference points from the less intense general radiation resulting in a Laué pattern superimposed on a monochromatic pinhole photograph. The Laué spots aid, rather than complicate, the interpretation of the structure. In one way, however, this type of photograph might be misleading as the Laué spots on the concentric rings furnish intensity maxima which would not appear on a true Laué or monochromatic pinhole pattern.

Figure 3 (*c*) (of the satisfactory copper) indicates that the orientation is random. Little evidence is to be seen of concentration of diffracted spots. An analysis of the photograph of the copper which formed ears, however, indicates preferred orientations (Fig. 3 (*d*)); the 001 plane of many crystals is in the rolling surface and a 100 direction is parallel to the rolling direction. This is evident from the four-fold symmetry of the pattern, together with the observation that the Laué interferences make angles of 0, 18, 45, 72, and 90 deg. with the rolling direction on the film, corresponding to reflections from the 100, 311, 331, 131, and 010 crystallographic planes, which are the most important reflecting planes of the face-centered cubic lattice when the X-ray beam is incident along the 001 direction. The most prominent intensity maxima occur on the 311 ring, making angles of 18 and 72 deg. with the rolling direction as required by the structure previously defined.

PROBABLE EXPLANATION OF EAR FORMATION

We are aware of the controversy regarding the nature of the stresses involved in drawing, that is, whether they are mainly compressive at right angles to the direction of extension or tensile along this axis. In the sheet having the structure described, four sets of 111 planes lie in position favorable for easy glide in 110 directions under the influence of either type of force, resulting in local elongation along both the rolling and transverse directions. At 45 deg. to these directions slip can occur on only two sets of 111 planes and these are less favorably located for slip. This difference in the ease of extension readily accounts for the formation of ears in the locations shown in the cups of Fig. 1.

Comparison of the Annealed Structures of Copper Rolled 70 and 99 per cent:

Although it is freely admitted that no precise statement can be made regarding the reason for the preferred orientation in the unsatisfactory copper, nevertheless, there is justification for the belief that it is largely due to the heavier reduction given this material prior to the last annealing. In order to obtain further knowledge regarding the significance of extreme rolling reductions, copper strips reduced 70 and 99 per cent were annealed under the same conditions at 525° C. The copper reduced 99 per cent showed extreme preferredness in the fiber diagram (Fig. 3 (*e*)), while the material rolled 70 per cent possessed a random orientation. (Fig. 3 (*f*)).

It is evident from the results obtained in this study together with the data of other investigators that there is a critical reduction, beyond which a fiber structure may be expected after annealing at about 525° C.[1] Inasmuch as preferredness may result from either (or both) excessive reductions and extremely high annealing temperatures it is obvious that a suitable mill schedule must include a proper correlation of these variables.

CONCLUSIONS

1. X-ray studies have demonstrated that hard-rolled copper has a pronounced fiber structure. In the sheets examined (rolled 70 and 99 per cent) it was found that the common orientation was char-

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[1] The authors are investigating the influence of other reductions on the structure of annealed copper.

acterized by a predominance of dodecahedral (110) planes in the surface and a $\bar{3}53$ direction along the rolling axis. Other investigators have reported a 112 direction for rolled copper.

2. A preferred orientation was found in the annealed sheet which formed ears when cupped. The copper which drew without ears showed random orientation.

3. Evidence has been furnished which indicates that preferred orientations in copper, annealed at the usual temperatures, may be attributed in many cases to excessively high rolling reductions.

4. It is suggested that the formation of ears in drawn copper may be avoided by limiting the reductions to approximately 65 per cent and annealing at temperatures ranging from 500 to 600° C.

DISCUSSION

Mr. C. H. Davis.[1]—I wish to present a photomicrograph that may be of some interest as shown in the accompanying Fig. 1. It is of cupro-nickel (85 per cent copper – 15 per cent nickel), taken at 75 diameters. You will note the two sets of twinning bands at about 90 deg. to each other, and at 45 deg. to the direction of rolling. This metal when cupped formed very prominent ears along the direction of rolling and at 90 deg. thereto.

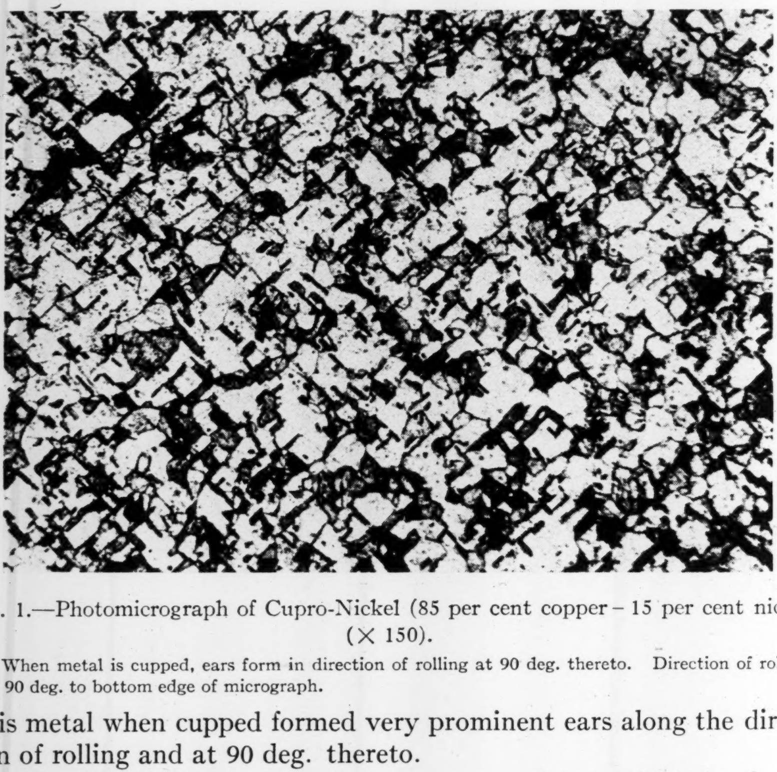

Fig. 1.—Photomicrograph of Cupro-Nickel (85 per cent copper – 15 per cent nickel (× 150).

When metal is cupped, ears form in direction of rolling at 90 deg. thereto. Direction of rolling is at 90 deg. to bottom edge of micrograph.

Mr. E. H. Dix, Jr.[2]—From our experience with aluminum alloys, I wonder if the purity of these two samples could be slightly different. It is given as 99.96 per cent, but in looking at the micrographs there seems to be in the micrograph of the sample which formed ears, slightly more insoluble constituent, which I assume is shown by the black particles, and this constituent seems to be slightly more definitely aligned than in the other sample. I wonder if the small difference in the amount of insoluble constituent might not have some effect on the directional properties.

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[1] Assistant Metallurgist, American Brass Co., Waterbury, Conn.

[2] Metallurgist, Research Bureau, Aluminum Co. of America, New Kensington, Pa.

THE CHAIRMAN (*Past-President H. F. Moore*[1]).—In conversation with Mr. McAdam concerning this paper the suggestion was brought out that possibly some of the phenomena of directional properties of cold-worked copper described in the paper may account for the conflicting results on the fatigue strength of cold-drawn copper obtained at Annapolis and at Illinois. At Annapolis cold-drawing was found to increase the endurance limit of copper, while at Illinois cold-drawn copper developed no higher endurance limit than did annealed copper. The tests at both places were made before any of the recent advances in X-ray analysis of atomic structure.

MESSRS. ARTHUR PHILLIPS[2] AND GERALD EDMUNDS[3] (*authors' closure by letter*).—The micrograph submitted by Mr. Davis is an interesting example of an annealed structure which showed pronounced directional properties on cupping. The general alignment of the twin bands at approximately 45 deg. to the direction of rolling is to be expected in cupro-nickel which has a face-centered cubic lattice.

With respect to the significance of the cuprous oxide content, we believe from observations made on other copper that the very slight difference in the amount of oxide in the two samples cannot account for the marked difference in the cupping properties described. In this connection, however, we hope to make further studies on the directional properties in wrought electrolytic and deoxidized copper.

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[2] Associate Professor of Metallurgy, Yale University, New Haven, Conn.

[3] Graduate Student, Department of Mining and Metallurgy, Yale University, New Haven, Conn

BEARING BRONZES WITH AND WITHOUT ZINC[1]

By H. J. French[2] and E. M. Staples[3]

Synopsis

This paper describes the results of laboratory tests of bearing bronzes with the object of classifying the alloys in the copper corner of the copper-tin-lead system according to the character of service for which they seemed to be best adapted. The effect of 4 per cent of zinc on the properties of these bronzes is also discussed.

Bronzes with less than about 4 per cent of tin were considered to be unsuited for general bearing service but those low in tin and high in lead should serve satisfactorily for special requirements involving low loads.

Bronzes with less than about 5 per cent of lead appeared to be suited only for service where lubrication could be maintained; those containing more than about 5 per cent of lead were best able of any of the groups studied to operate for short periods in the absence of lubrication.

In general, the addition of 4 per cent of zinc to the copper-tin-lead alloys had a small influence upon the properties of the bronzes studied and with few exceptions such changes as were observed seemed to be beneficial rather than detrimental for bearing service.

Further development of test methods may show that zinc tends to increase the wear losses and duration of the "wearing-in" period of bronze bearings, but since this is also a function of the perfection of fit such effects would be disadvantageous only in certain cases.

The results seem to justify the conclusion that the effects of zinc up to 4 per cent are generally small and may be insignificant in comparison with changes in properties quite readily produced in bronzes from variations in foundry practice.

Introduction

Within the past three years an extended study was made, in cooperation with The Chicago Bearing Metal Co. and the Magnus Co., of Chicago, Ill., of the wear and mechanical properties of copper-tin-lead alloys widely used in railroad bearings. As already reported[4] this study was made with the two-fold purpose of developing a laboratory testing technique for bearing metals and of finding reasons for the wide variations in the specifications of different carriers for bearings for similar conditions of service.

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[1] Published with the approval of the Director of the Bureau of Standards of the U. S. Department of Commerce, Washington, D. C.

[2] Senior Metallurgist, U. S. Bureau of Standards, Washington, D. C.

[3] Research Associate of The Bunting Brass and Bronze Co. at the U. S. Bureau of Standards, Washington, D. C.

[4] H. J. French, S. J. Rosenberg, W. LeC. Harbaugh and H. C. Cross, "Wear and Mechanical Properties of Railroad Bearing Bronzes at Different Temperatures," U. S. Bureau of Standards *Journal of Research*, Vol. 1, No. 3, September, 1928; also H. J. French, "Wear and Mechanical Tests of Some Railroad Bearing Bronzes," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 298 (1928).

The testing technique developed in the investigation did not define completely all those properties of importance for alloys intended for service in bearings, but the methods used can properly form part of a more complete testing technique since they gave information consistent with practical experience. Therefore, more extended application is justified.

Comparisons of the different alloys were based on wear tests, single blow impact tests on notched bars, repeated pounding tests, tension or hardness tests and microscopic examination. The wear tests were made in the Amsler wear testing machine and in addition to determination of weight losses, attention was given to the frictional properties (torque in the tests) and the character of the worn surfaces. Wear tests were made in the presence of a lubricant at atmospheric temperatures, and tests were also made without lubrication at atmospheric and elevated temperatures up to and including 350° F. (175° C.). With the exception of the hardness tests, made only at atmospheric temperatures, the various mechanical tests were carried out at temperatures up to and including 600° F. (315° C.).

TABLE I.—COMPARISON OF TWO SPECIFICATIONS FOR THE BRONZES CONTAINING 80 PER CENT COPPER, 10 PER CENT TIN AND 10 PER CENT LEAD.

Element	A.S.T.M. Specifications (B 74 - 28 T)	Illinois Central Railroad Co. Specifications
Copper, per cent	79 to 81	79 to 81
Tin, per cent	9 to 11	9 to 11
Lead, per cent	9 to 11	9 to 11
Zinc, per cent	max., 0.25	1 to 3
Iron, per cent	max., 0.10	....
Nickel, per cent	max., 0.50	....
Phosphorus, per cent	max., 0.25	max., 0.25
Aluminum, per cent	none	....
Sulfur, per cent	max., 0.05	....
Antimony, per cent	max., 0.25	....
Other elements, per cent	max., 0.15	max., 0.50

The copper-tin-lead alloys previously studied were made from commercially pure raw materials, but it is seldom practicable for economic reasons to produce bearing bronzes on a commercial basis wholly from new or "virgin" metals. The largest part of the bearings now manufactured are made of remelted bearings combined with varying amounts of shop scrap and only sufficient new metal to bring the mixtures to the required compositions.

The scrap charged in the form of unserviceable bearings is generally gathered from a multitude of sources and not only varies in the proportions of copper, tin and lead but also in the proportions of impurities present, such as zinc, antimony, phosphorus, iron, or nickel, etc. The subject of impurities is then of primary importance in the commercial production of bearing bronzes, and specifications indicate a wide difference of opinion with respect to the maximum allowable limits for different elements. An example is found in comparison of two specifications for the well-known alloy containing 80 per cent of copper, 10 per cent of tin and 10 per cent of lead. As shown in Table I, the Society's Tentative Specifications B 74 – 28 T for sand castings[1] permit a maximum of 0.25 per cent of zinc while the specifications of a prominent carrier call for a zinc content between 1 and 3 per cent.

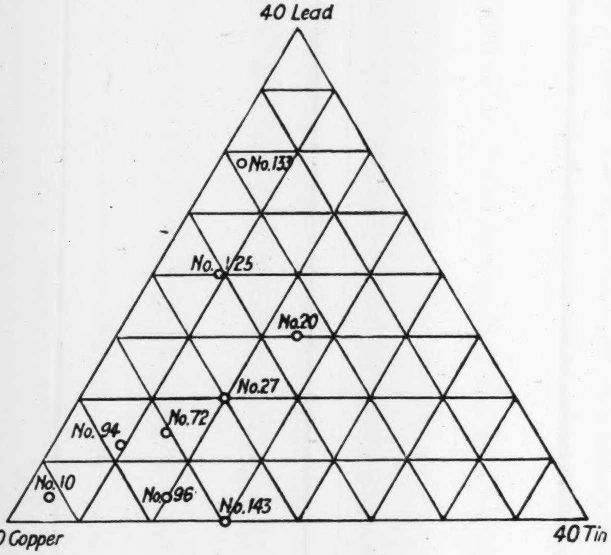

FIG. 1.—Ternary Diagram Showing Location of Bronzes Tested.

Lack of information on the effects of common impurities and the wide variations in the proportions of copper, tin, and lead specified by different purchasers for bearing metals intended for similar service resulted in the establishment of a research associateship at the U. S. Bureau of Standards by The Bunting Brass and Bronze Co., Toledo, Ohio, for the purpose of studying the properties and applications of bearing bronzes. The tests here reported constitute the first phase of a study of this subject and relate to the properties of copper-tin-lead alloys widely used in bushings in the automotive and other industries and to the effect of zinc on such bronzes.

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[1] *Proceedings,* Am. Soc. Testing Mats., Vol. 28, Part I, p. 766 (1928); also 1928 Book of A.S.T.M. Tentative Standards, p. 119.

TABLE II.—CHEMICAL COMPOSITIONS OF THE BRONZES TESTED.

Alloy	Chemical Composition[c]					Used for Wear Tests, *W*, or Mechanical Tests, *M*
	Copper, per cent	Tin, per cent	Lead, per cent	Zinc, per cent	Total, per cent	
No. 10[a]	96.5	1.8	1.8	...	100.1	W
No. 10	95.4	2.1	2.0	...	100	M
No. 10Z[a]	91.4	2.0	1.9	4.5	99.8	W
No. 10Z	92.5	2.0	1.8	3.7[b]	100	M
No. 20[a]	73.7	11.8	14.4	...	99.9	W
No. 20	73.1	12.8	14.1	...	100	M
No. 20Z[a]	69.4	12.1	15.1	3.2	99.8	W
No. 20Z	70.5	11.9	13.8	3.8[b]	100	M
No. 27	80.4	10.0	9.5	...	99.9	W
No. 27	79.8	10.3	9.6	...	99.7	M
No. 27Z	76.5	9.4	9.8	4.1	99.8	W
No. 27Z	76.4	10.1	9.1	4.4[b]	100	M
No. 27[a]	80.5	9.8	9.6	...	99.9	W
No. 27Z[a]	77.4	9.0	10.1	3.3	99.8	W
No. 72	85.8	7.3	6.8	...	99.9	W
No. 72	85.3	7.3	7.2	...	99.8	M
No. 72Z	82.2	6.8	7.0	4.0	100	W
No. 72Z	82.5	7.2	6.7	3.6[b]	100	M
No. 94	89.3	4.5	6.2	...	100	W
No. 94	89.6	4.8	5.6	...	100	M
No. 94Z	85.3	4.8	5.8	4.1	100	W
No. 94Z	85.6	4.6	5.8	4.0[b]	100	M
No. 96	88.1	10.0	2.0	...	100.1	W
No. 96	88.4	10.3	1.5	...	100.2	M
No. 96Z	83.9	9.3	2.1	4.4	99.7	W
No. 96Z	84.8	9.5	2.0	3.7[b]	100	M
No. 96[a]	88.5	9.5	2.0	...	100	W
No. 96Z[a]	84.8	9.9	2.1	3.1	99.9	W
No. 125	75.3	4.5	20.1	...	99.9	W
No. 125	76.6	4.6	18.9	...	100.1	M
No. 125Z	72.8	4.3	19.1	3.7	99.9	W
No. 125Z	73.0	4.3	18.5	4.2[b]	100	M
No. 133[a]	68.8	2.4	28.7	...	99.9	W
No. 133	75.2	1.5	23.2	...	99.9	M
No. 133Z[a]	66.3	2.2	27.8	3.7	100	W
No. 133Z	71.0	1.4	23.8	3.8[b]	100	M
No. 143	85.2	14.7	....	...	99.9	W
No. 143	84.8	15.0	....	...	99.8	M
No. 143Z	81.1	14.5	....	4.4[b]	100	W
No. 143Z	82.0	13.8	....	4.2[b]	100	M

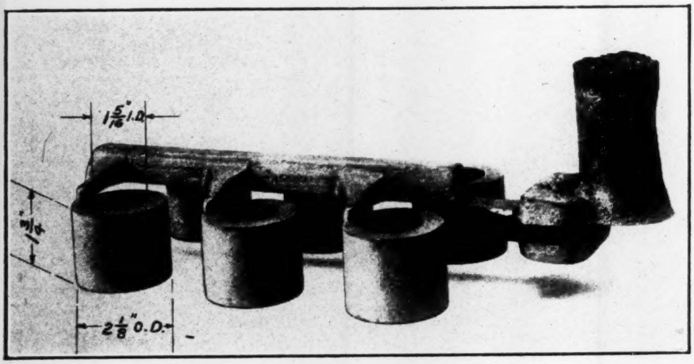
[a] These castings in addition to those for the mechanical tests were made at the U. S. Bureau of Standards; all others were made in the foundries of The Bunting Brass and Bronze Co., Toledo, Ohio.

[b] Zinc by difference.

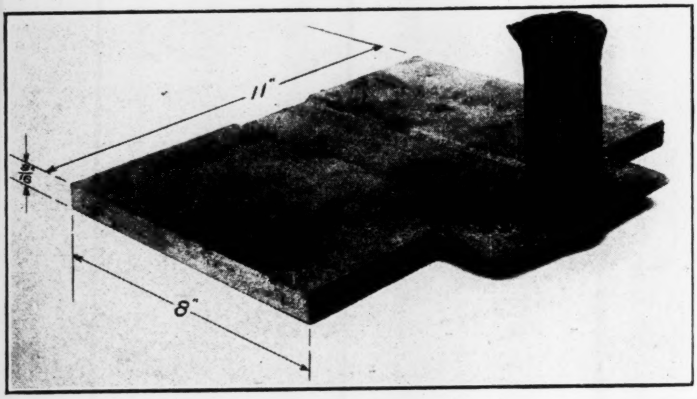
[c] Twenty-four of the samples were analyzed for phosphorus and antimony; 12 were analyzed for iron and sulfur; in no case was any one of these elements present in quantities greater than 0.02 per cent.

ALLOYS STUDIED

Most of the compositions tested were selected from the group regularly manufactured by The Bunting Brass and Bronze Co. Some other alloys were included to secure a more general distribution over the copper corner of the copper-tin-lead system and thus permit comparisons on the basis of ternary diagrams. However, in most cases due regard was given, in the selections, to the industrial production requirements.



(*a*) Wear-test specimens.



(*b*) Impact, pounding and hardness-test specimen.

FIG. 2.—Test Castings Used.

Two groups of alloys were prepared. The first comprised alloys of copper, tin and lead, of compositions shown in Fig. 1. In the second group the copper-tin-lead ratios were kept the same and 4 per cent of zinc was added.

It was, of course, not practicable to adhere exactly to the intended compositions in the production of the test castings, but the deviations encountered can be neglected since comparisons are based largely upon the trends in the copper-tin-lead and copper-tin-lead-zinc systems. The chemical compositions of the different test castings are summarized in Table II.

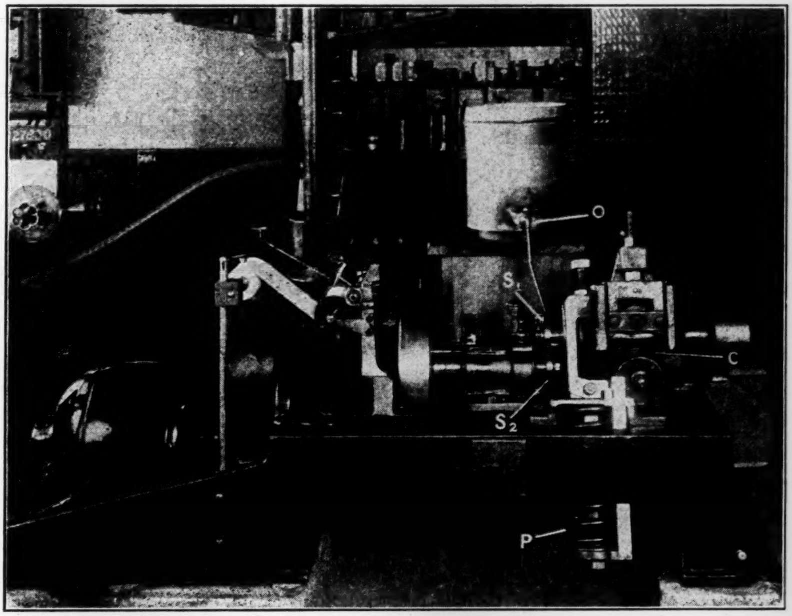


FIG. 3.—Amsler Wear Testing Machine.

Surfaces of specimens S_1 and S_2 move in the same direction but at different speeds, with lateral oscillation produced by cam *C* and under contact pressures controlled by spring *P*. The friction is recorded on the torque indicator *T*. In tests with lubrication the oil is supplied from reservoir *O*. In all tests the total load between specimens was 37.5 lb.; the slip, 12 ft. per min.; the amplitude of lateral oscillation, 0.3 in.

PREPARATION OF THE TEST CASTINGS

The test castings were of relatively thin sections to give structures and properties comparable to those found in the small bushings so widely used in the automotive industries. Castings for the wear test specimens were hollow cylinders while those used for the impact, pounding and hardness test specimens were in the form of plates as is shown in Fig. 2.

The method of preparation of the castings was substantially as follows. Approximately 1400 lb. of copper (trolley) wire was melted in a gas-fired furnace under a covering of limestone and borax. About 8 per cent of tin was added to the copper, when melted, and after thorough stirring the metal was cast into small notched ingots which were used as the basis of the melts for the test castings. These ingots were melted in a small gas-fired crucible furnace together with the required amounts of copper, tin and lead under a limestone-borax slag. Zinc, when required, was added after skimming the metal. All the castings were made in sand molds. Casting temperatures were measured with chromel-alumel thermocouples and a potentiometer.

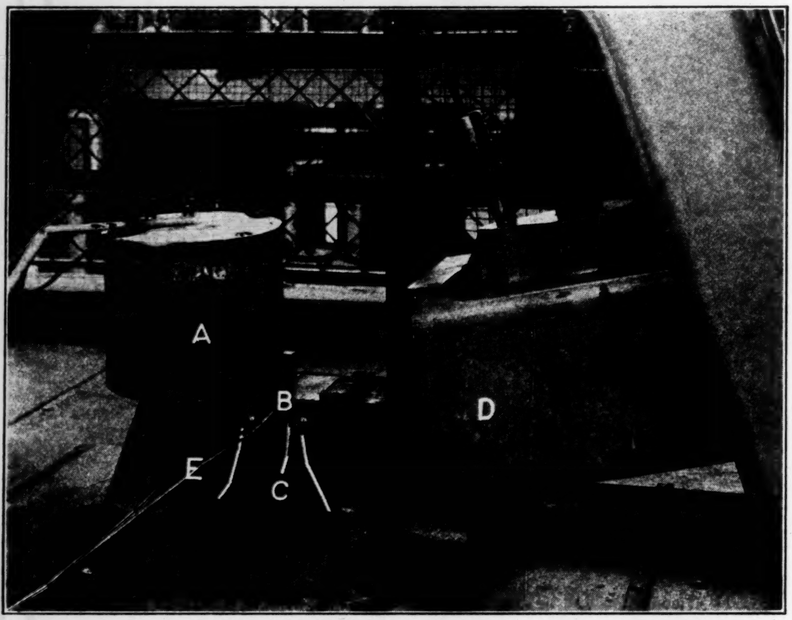


FIG. 4.—Part of the Equipment Used in the Notched-Bar (Izod) Impact Tests.

Specimen *B* is subjected to a blow from tup *D* immediately after removal of equalizing furnace *A*. The coils, *C*, permit the vise to be brought to the temperature of furnace *A*.

The metal was poured at approximately 2000° F. (1095° C.) except in the case of alloys Nos. 10 and 133 (Table II). A somewhat higher pouring temprature, around 2100° F. (1150° C.), was required to fill the molds with sound metal with alloy No. 10 containing 96 per cent of copper and 2 per cent each of tin and lead.

The large amount of lead (28 per cent) in alloy No. 133 made necessary the use of a somewhat lower pouring temperature, estimated around 1950° F. (1065° C.), in order that the copper-tin solution would freeze in the mold before the lead had a chance to segregate to any great degree. In all cases the metal was stirred continuously from the time the crucible was removed from the furnace until the metal was ready to pour.

METHODS OF TEST

Metals used as bearings are subjected to a wide range of conditions in service. Failures occur by wear, or by inability to carry the required loads, to withstand impact or to resist deformation ("pounding out"). In addition to these properties, consideration should be given to the frictional properties (for example, starting torque), ability to operate at elevated temperatures, ease of manufacture and other characteristics.

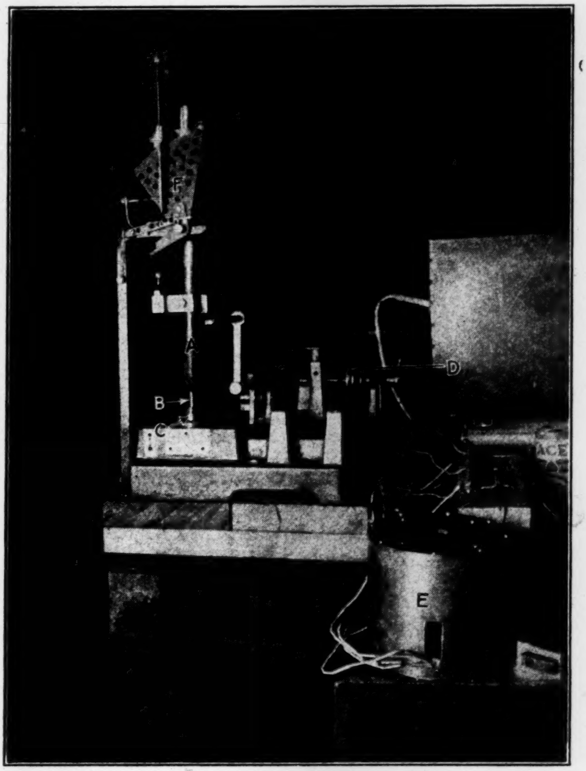


FIG. 5.—Equipment Used in the Repeated Pounding Tests.

Specimen *B*, on anvil *C*, is subjected to repeated blows in compression by the falling weight *A*. Furnace *E*, mounted on anvil *C*, is used for tests at elevated temperatures. *D* is the driving mechanism and *F* prevents auxiliary blows from rebound of *A*.

Tests made with a weight of 7.15 lb. falling through a distance of 2 in.

The conditions encountered in practical service are seldom simple. Most often they are such that the selection of the bearing metal is a compromise to obtain the best combination of properties. The development of a testing technique which permits a logical selection for the varied conditions of practical service is therefore not simple. The situation is further complicated by the fact that the study of the wear of bearing metals cannot be divorced from that of lubrication since most bearings are designed for service in the presence of oils, greases or other lubricants. The conditions of lubrication encountered industrially vary from so-called complete film lubrication to practically no lubrication at all.

In this investigation wear tests were made both in the presence and absence of oil, as these relate to extreme conditions. The resistance to deformation was studied by repeated pounding tests and the brittleness by single blow impact tests on notched bars. Since bearings are often required to operate at temperatures above atmospheric, the different tests were made at elevated as well as at atmospheric temperatures. These methods of test are the same as those previously used and have already been described in detail.[1] Photographs of the equipment used together with brief descriptions of the principles of operation are given in Figs. 3, 4, and 5.

Objections have been raised to the described methods of test on the basis that correct design can make the properties of the metals of construction for bearings of minor importance. Some engineers usually also favor tests of full-size bearings under practical conditions of lubrication and question the usefulness of wear tests made without lubricants. The properties of the metals of construction for bearings probably are of minor importance in cases where complete film lubrication can be guaranteed and where also design can reduce service stresses. However, the achievement of this ideal is not always practicable for economic or other reasons and the properties of the metals of construction still demand consideration. While the selected methods of test do not completely characterize those properties of interest in bearing metal applications and probably cannot be justly used for fine discriminations they should enable classification of groups of metals in a way which will eliminate large numbers of alloys from consideration when attempting to select metals for definite types of service. Final selection can then be made from a smaller group by actual service tests or in other ways. In fact the data to be discussed in this report have already been used successfully in this manner.

While questions relating to methods of test were discussed at some length in the report of preliminary work already referred to, it may be well to point out again that two of the principal points of contention concerning the work to be reported relate to the usefulness of dry wear tests and the adaptability of the Amsler wear testing machine to the study of bearing metals.

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[1] H. J. French, S. J. Rosenberg, W. LeC. Harbaugh and H. C. Cross, "Wear and Mechanical Properties of Railroad Bearing Bronzes at Different Temperatures," U. S. Bureau of Standards *Journal of Research*, Vol. 1, No. 3, September, 1928; also H. J. French, "Wear and Mechanical Tests of Some Railroad Bearing Bronzes," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 298 (1928).

As has been pointed out already, the conditions of lubrication encountered industrially vary from so-called complete film lubrication to practically no lubrication at all. Quite probably much of the wear produced in practical service occurs during periods of so-called boundary lubrication. Since this is essentially an unstable condition of lubrication it does not appear practicable at this stage of the development of wear testing to obtain reproducible results under conditions of boundary lubrication. Wear is not an important factor in the presence of complete film lubrication, and, therefore, it seems logical to use tests without lubricants as the base line for present comparisons and the development of a more complete testing technique. Furthermore, the results so far obtained have in most respects been generally consistent with practical experience.

TABLE III.—COMPOSITION AND HEAT TREATMENT OF THE STEEL USED IN THE WEAR TESTS.

CHEMICAL COMPOSITION:	
Carbon, per cent	0.93
Manganese, per cent	1.11
Silicon, per cent	0.20
Chromium, per cent	0.32
Tungsten, per cent	0.36

HEAT TREATMENT:
1500° F. (815° C.), 30 min. quench in thin oil;
350° F. (175° C.), 1 hour quench in thin oil.

HARDNESS AFTER TREATMENT:
Rockwell "C" Scale 61 to 63

The selection of the Amsler wear testing machine, in which it is practicable to approach line contact between the bearing and shaft metals, is justified by the elimination of the factor of fit. In the suggested tests of full-size bearings it is difficult to eliminate variations from the factor of fit, by which is meant variations in the initial smoothness of the contact surfaces and how well these conform one to the other. It is known that variations from these sources can materially affect the amounts and rates of wear and the results of tests of full-size bearings are more often indicative of the manner of installation or the conditions of lubrication than of the performance of the metals.

Further development and perfection of the methods of test used in this investigation can be expected to throw further light upon justifiable interpretations of the results, but for the present the principal use of the data, and the only one recommended, is the grouping of the metals according to the broad classes of service for which they are best adapted.

The steel specimens which, for the previous wear tests, were cut from used medium-carbon-steel railroad axles were replaced by hardened steel of the composition and heat treatment shown in Table III.

The selection of this steel was the result of a careful survey by The Bunting Brass and Bronze Co. of the applications of their bushings. A wide difference was found in the shafting materials, but in a majority of cases hardened steels were employed. Case-hardened low-carbon steels were most often used, but as difficulties were anticipated in reproducing results with case-hardened low-carbon steel specimens, the oil hardening steel (Table III) was selected. This selection was based on comparable surface hardness and is considered to represent the steel contact surfaces encountered in very many of the practical applications of the bearing bronzes studied.

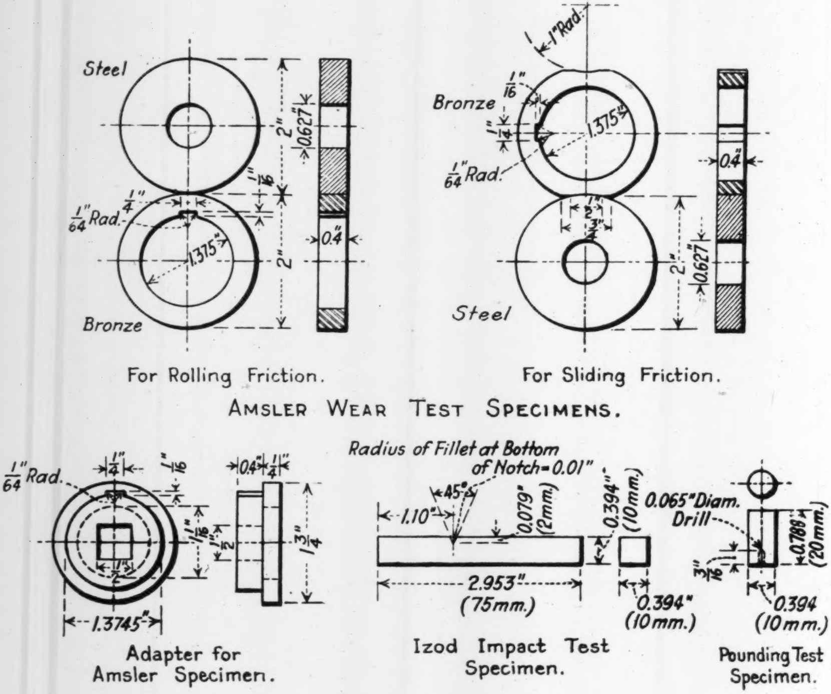
Steel
Bronze
2"
0.627"
1/4"
1/16"
1/64" Rad.
1.375"
0.4"
1" Rad.
For Rolling Friction.
For Sliding Friction.
AMSLER WEAR TEST SPECIMENS.
1/64" Rad.
1/4"
1/16"
0.4"
1 3/4"
1/2"
1.3745"
Radius of Fillet at Bottom of Notch = 0.01"
45°
1.10"
0.079" (2mm)
0.394" (10mm.)
2.953" (75mm.)
0.394" (10mm.)
0.065" Diam. Drill
0.788" (20mm.)
3/16"
0.394 (10mm.)
Adapter for Amsler Specimen.
Izod Impact Test Specimen.
Pounding Test Specimen.

FIG. 6.—Test Specimens.

The test specimens used in the different tests are shown in Fig. 6. They are similar to those employed in previous work with the exception of the specimen for the repeated pounding tests which had a ratio of length to diameter of 2 instead of 2.5. This change reduced bending of the specimens and resulted in more consistent results at large deformations.

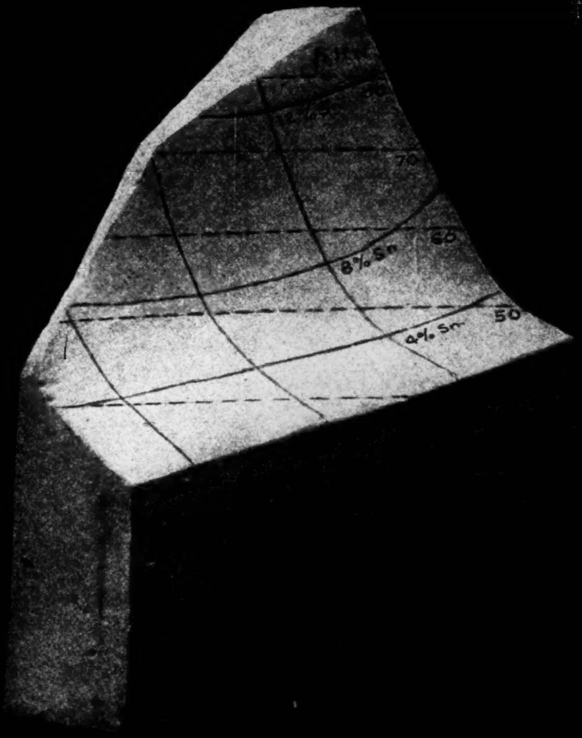

FIG. 7.—Approximate Brinell Hardness Values for Part of the System Copper-Tin-Lead.

Based on tests with a 10-mm. ball and a 500-kg. load.

EXPERIMENTAL RESULTS

Hardness and Structure of the Copper-Tin-Lead Alloys:

The tests on the copper-tin-lead alloys, without zinc, confirmed the results of previous tests[1] on a narrower range of compositions and make possible a classification of alloys over a considerable area of the

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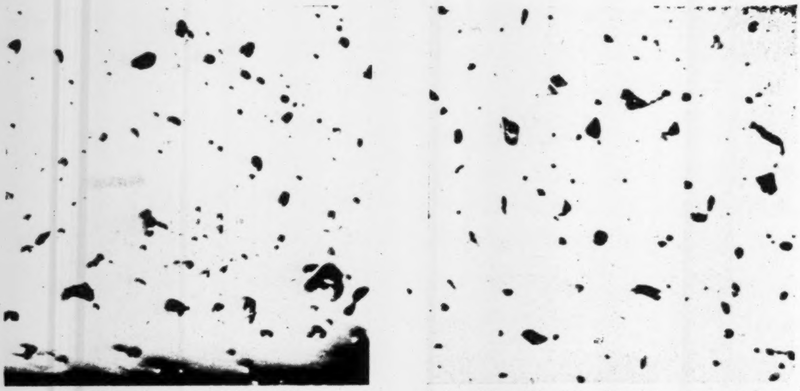
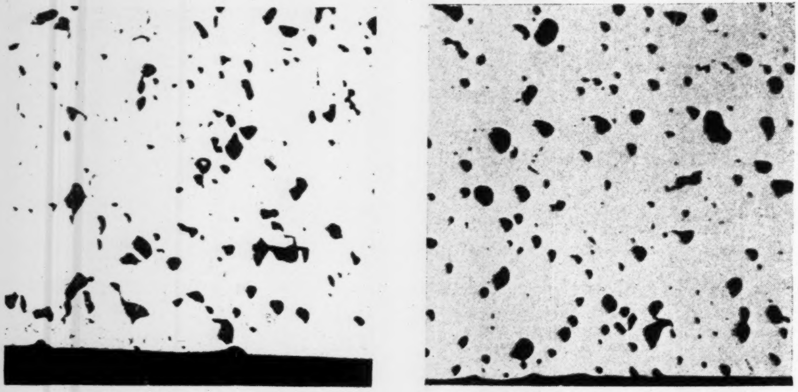
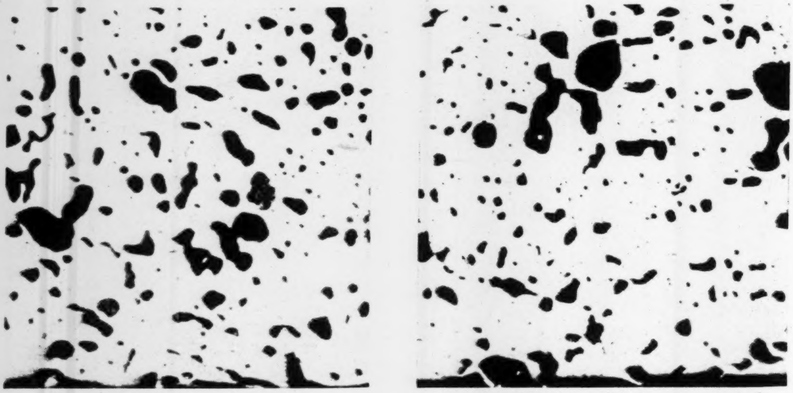
[1] *Loc. cit.*



(*a*) Alloy No. 10, without zinc. (*b*) Alloy No. 10, with 4 per cent of zinc.



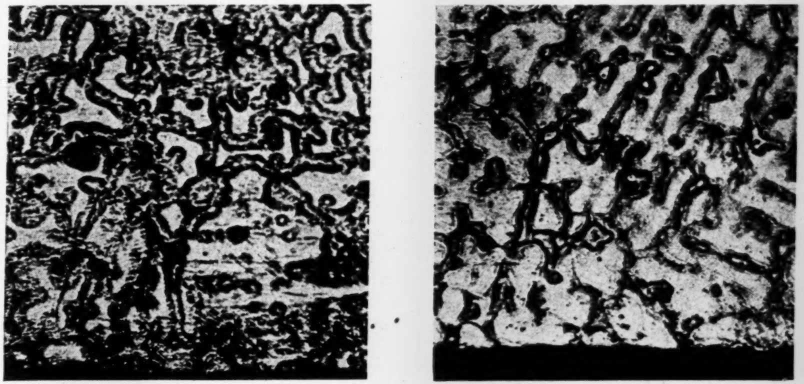
(*c*) Alloy No. 72, without zinc. (*d*) Alloy No. 72, with 4 per cent of zinc.



(*e*) Alloy No. 125, without zinc. (*f*) Alloy No. 125, with 4 per cent of zinc.

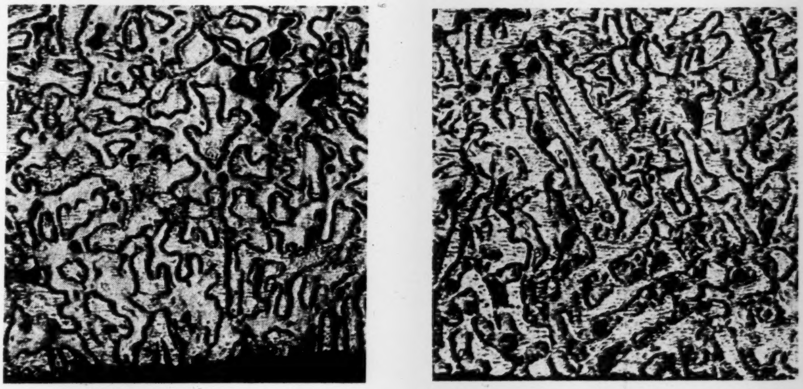
FIG. 8.—Unetched Sections of Some of Bronzes Tested (× 100).

Alloy No. 10—Nominally 96 per cent copper, 2 per cent tin, 2 per cent lead.
Alloy No. 72—Nominally 85.4 per cent copper, 7.3 per cent tin, 7.3 per cent lead.
Alloy No. 125—Nominally 75.5 per cent copper, 4.5 per cent tin, 20.0 per cent lead.



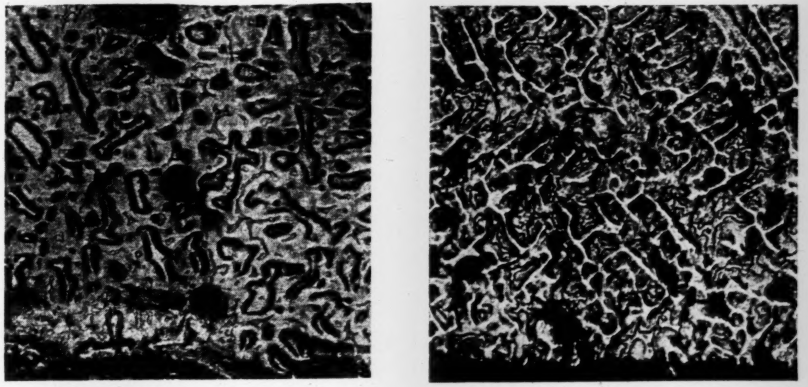
(*a*) Alloy No. 94, without zinc.

(*b*) Alloy No. 94, with 4 per cent of zinc.



(*c*) Alloy No. 72, without zinc.

(*d*) Alloy No. 72, with 4 per cent of zinc.



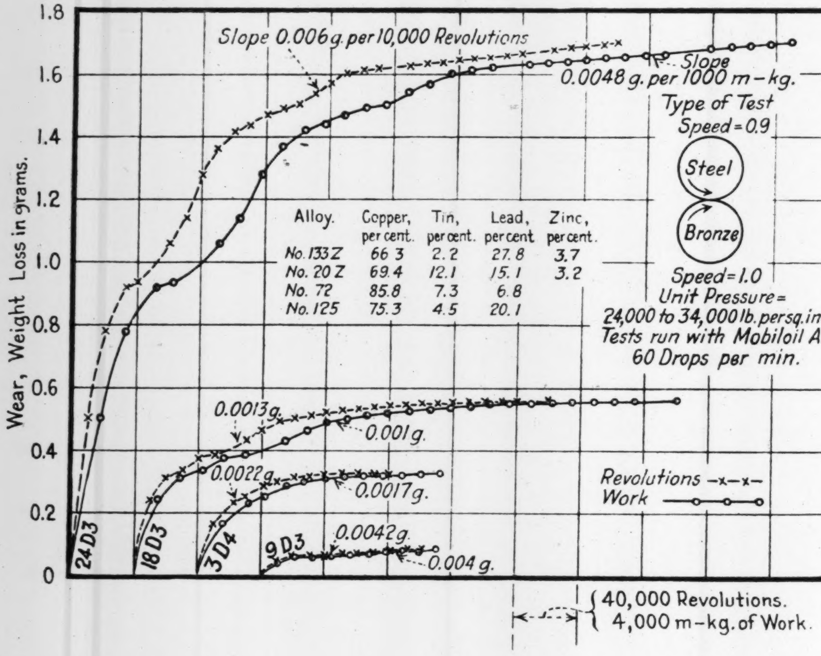
(*e*) Alloy No. 27, without zinc.

(*f*) Alloy No. 27, with 4 per cent of zinc.

FIG. 9.—Micrographs of Some of the Bronzes Tested (× 100). Etched with solution of 3 parts NH_4OH plus 1 part H_2O_2 followed by solution of $FeCl_3$ in HCl (10 g. $FeCl_3$ plus 30 cc. concentrated HCl plus 120 cc. H_2O).

Alloy No. 94—Nominally 89.4 per cent copper, 4.5 per cent tin, 6.1 per cent lead.
Alloy No. 72—Nominally 85.4 per cent copper, 7.3 per cent tin, 7.3 per cent lead.
Alloy No. 27—Nominally 80.0 per cent copper, 10.0 per cent tin, 10.0 per cent lead.

copper corner of the copper-tin-lead ternary diagram. Solid models are used as the basis for discussion since these offer a convenient means of indicating the trends produced by combined variations in the proportions of copper, tin and lead. In these models the position of any point on the triangular base represents the chemical composition while the height represents the particular property under discussion. General trends are shown without emphasizing minor variations which are of no great practical importance and are probably associated with experimental errors.



Alloy.	Copper, per cent.	Tin, per cent.	Lead, per cent	Zinc, per cent.
No. 133Z	66 3	2.2	27.8	3.7
No. 20Z	69.4	12.1	15.1	3.2
No. 72	85.8	7.3	6.8	
No. 125	75.3	4.5	20.1	

FIG. 10.—The Relations Between Wear, Work and Revolutions in Wear Tests Made in the Presence of Oil.

As shown in Fig. 7, the hardness of the bronzes increased with increase in the proportions of tin, particularly when the alpha solid solubility of tin in copper was exceeded. For the conditions encountered in the preparation of the test castings, the limit of this solubility was at about 9 per cent tin in copper as will be evident from examination of the micrographs of Fig. 9. As the lead in the bronzes increased the hardness decreased, whether the bronzes contained large or small proportions of tin. These characteristics are well known and

Figs. 7, 8, and 9 are given merely as a record of the hardness and structures of the particular castings tested.

Wear With and Without a Lubricant:

Before discussing the results of other tests, it should be recalled that difficulties were encountered in earlier work in attempting to define the wearing properties of bronzes by tests in the presence of lubricants. These difficulties were due partly to the fact that with what was probably complete film lubrication the metals were not in continuous and direct contact and the wear dropped to such low rates after an initial "wearing-in" period that it became impracticable to differentiate with any degree of certainty between the various metals. As previously stated, there are not at present available well-defined

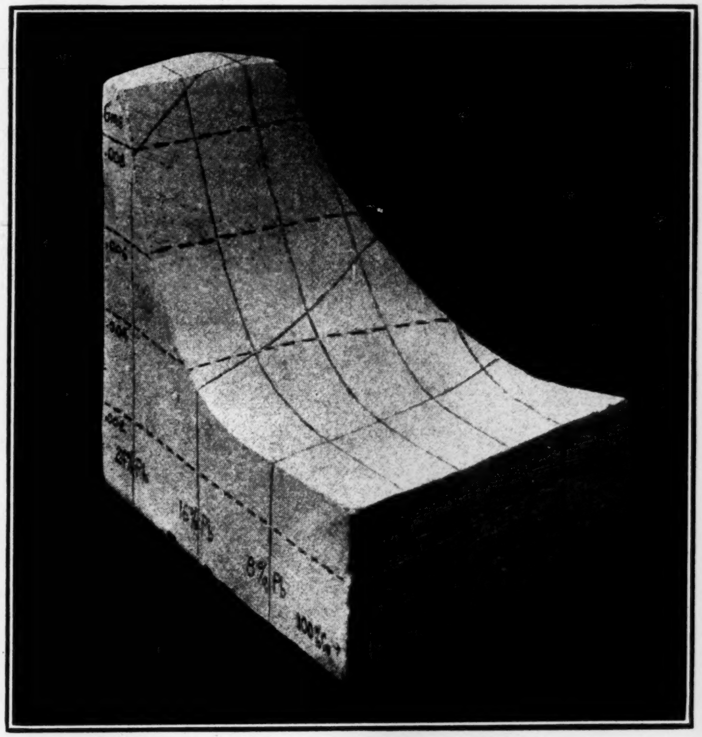


FIG. 11.—Wear (Subsequent to the "Wearing-in" Period) of Bronzes Tested with Lubrication at Atmospheric Temperatures.

Wear expressed in weight loss per 10,000 revolutions. Conditions of test, including the oil used, are described in detail in the reference given in the footnote on page 450.

methods for making wear tests under the unstable conditions of so-called "boundary lubrication" where wear is liable to occur in practical service, and accordingly the wear tests upon which the major comparisons were based were made dry, that is, in the absence of the customary lubricants.

TABLE IV.—SUMMARY OF THE RESULTS OF THE WEAR TESTS WITH LUBRICATION.

Lot	Chemical Composition				Rate of Wear after Initial Period		Total Number[a] of Revolutions	Total[a] Work Done, m-kg.	Total[a] Weight Loss, g.
	Copper, per cent	Tin, per cent	Lead, per cent	Zinc, per cent	Weight Lost per 1000 m-kg., g.	Weight Lost per 10000 revolutions, g.			
BRONZES WITH LESS THAN 5 PER CENT LEAD									
No. 11D3..........	85.2	14.7	...	...	0.0019	0.0021	100 000	10 600	0.07
No. 11D4..........	85.2	14.7	...	...	0.0026	0.0033	160 000	19 000	0.45
No. 12D3..........	81.1	14.5	...	4.4	0.0027	0.0039	340 000	47 600	1.05
No. 7D3..........	88.1	10.0	2.0	...	0.0031	0.0033	120 000	14 600	0.15
No. 15D3..........	96.5	1.8	1.8	...	0.0028	0.0032	90 000	10 200	0.05
No. 16D3..........	91.4	2.0	1.9	4.5	0.0031	0.0042	330 000	43 800	0.89
Average.........	....	....	....	....	0.0027	0.0033			
BRONZES WITH FROM 5 TO 12 PER CENT LEAD									
No. 1D3...........	80.4	10.0	9.5	...	0.0025	0.0027	120 000	13 900	0.11
No. 3D3...........	85.8	7.3	6.8	...	0.0034	0.0042	120 000	14 300	0.25
No. 3D4...........	85.8	7.3	6.8	...	0.0017	0.0022	120 000	15 300	0.33
No. 5D3...........	89.3	4.5	6.2	...	0.0019	0.0025	120 000	14 700	0.14
Average.........	....	....	....	....	0.0024	0.0029			
BRONZES WITH 15 PER CENT OR MORE LEAD									
No. 17D3..........	73.7	11.8	14.4	...	0.0042	0.0052	160 000	19 300	0.51
No. 18D3..........	69.4	12.1	15.1	3.2	0.0010	0.0013	260 000	34 100	0.56
No. 9D3..........	75.3	4.5	20.1	...	0.0040	0.0042	100 000	11 100	0.08
No. 9D4..........	75.3	4.5	20.1	...	0.0042	0.0053	160 000	19 700	0.49
No. 23D3..........	68.8	2.4	28.7	...	0.0071	0.0089	120 000	13 200	0.25
No. 24D3..........	66.3	2.2	27.8	3.7	0.0048	0.0060	340 000	45 000	1.71
Average.........	....	....	....	....	0.0042	0.0052			

[a] When the tests were discontinued.

Such tests gave only a partial picture of the wearing properties of the bronzes tested in the previous investigation, and, therefore, further efforts were made to secure interpretable data from wear tests made in the presence of oil. Tests were continued for long periods and the slopes of the wear-revolution and wear-work curves subsequent to the "wearing-in" period (illustrated in Fig. 10) are used as the basis of Fig. 11.

However, these tests cannot justly be given as much weight as those made without oil, since the differences in duplicate determinations (Table IV) were more nearly of the order of magnitude of the differences between different bronzes than was the case in the dry wear tests. This is due to the fact that in the dry wear tests the bronzes quickly showed fairly constant and relatively high rates of wear, while in the presence of oil the first rapid wear was followed by decreases to very low wear rates (Fig. 10). Furthermore, while the tests with oil were continued for periods four to seven times as long as those required to reach an actual or apparent state of equilibrium in the dry wear tests, it is not known whether further decreases in the wear rates and a practical elimination of the differences between the different bronzes would result if the tests in the presence of oil were continued for even longer periods.

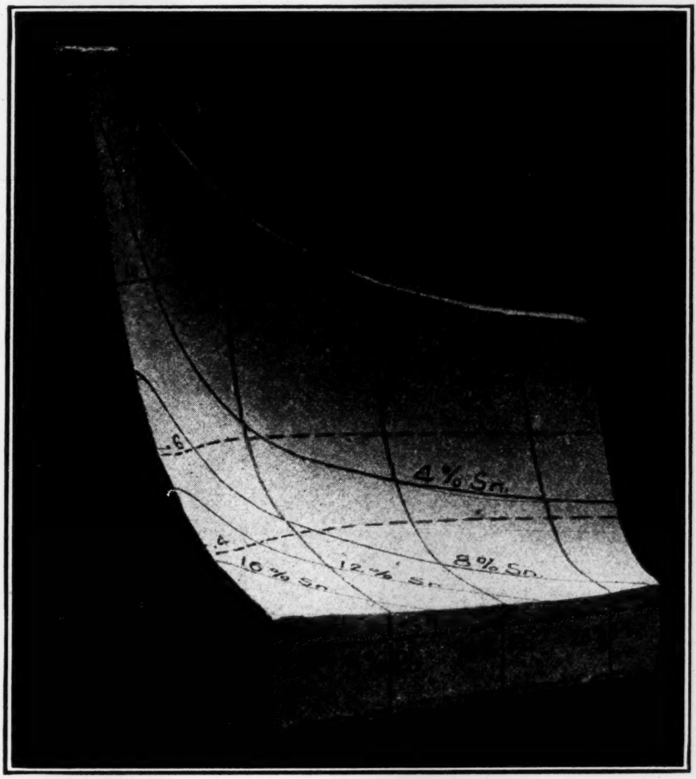
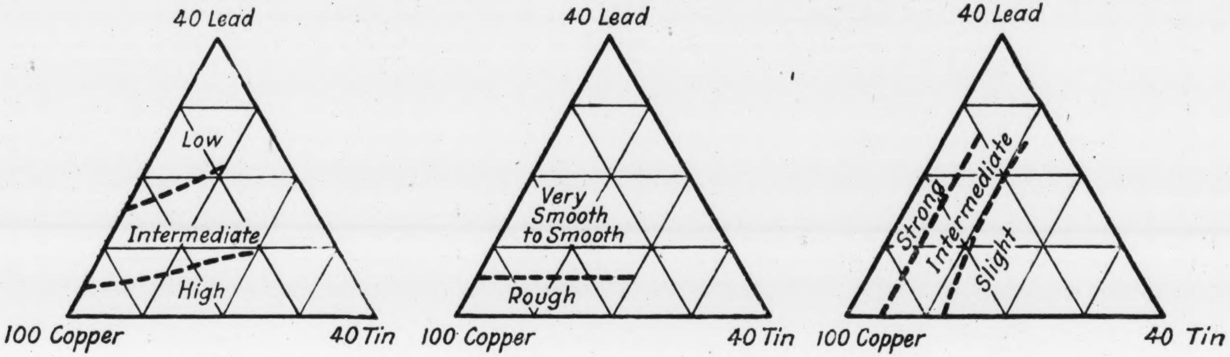


FIG. 12.—Wear of Bronzes Tested Without Lubrication at Atmospheric Temperatures.

Wear expressed in weight loss per 10,000 revolutions. Conditions of test are described in detail in the reference given in the footnote on page 450.

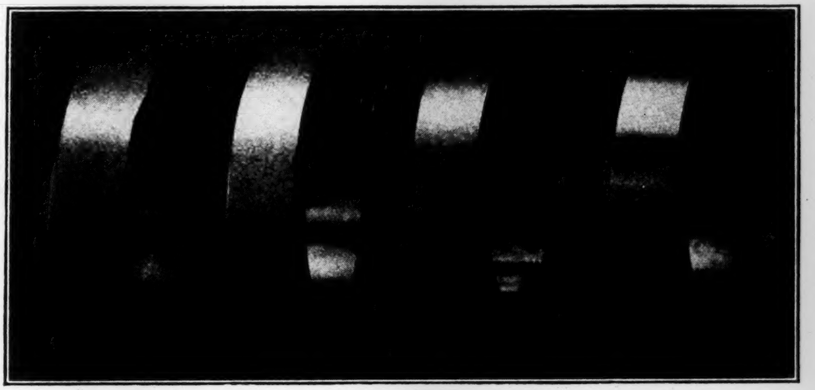


(*a*) Torque. (*b*) Character of worn surface. (*c*) Tendency toward "flat wheel" effect.

FIG. 13.—Special Features Observed in the Wear Tests Without Lubrication at Atmospheric Temperatures.

An adequate testing technique has not yet been developed for wear of bearing bronzes in the presence of lubricants and while the individual differences shown in Table IV were disregarded and the average wear rates of groups of bronzes made the basis of the smooth contours of the solid model shown in Fig. 11, some definite trends are shown which can justly be considered along with the results of the dry wear tests.

Aside from the very low rates of wear shown by all the bronzes subsequent to the "wearing-in" period, it will be observed that the high-lead bronzes wore somewhat more rapidly in the presence of oil than did the low-lead bronzes. This is directly opposite to the effects observed in the absence of oil.



(a) (b) (c) (d)

FIG. 14.—Photographs of Some of the Wear Test Specimens Before and After Test.

(a) "Flat wheel" condition in one of the bronzes tested (alloy No. 10Z).
(b) Bronze specimen showing rough type of wear (alloy No. 143).
(c) Bronze specimen showing smooth type of wear (Alloy No. 20).
(d) Bronze specimen before test (machine surface) (alloy No. 96).

The addition of 4 per cent of zinc to the copper-tin-lead alloys did not produce outstanding differences in the wear rates subsequent to the "wearing-in" period in the tests made with oil. However, the weight losses and the durations of the "wearing-in" periods were generally greater in the bronzes with zinc than in the corresponding alloys without zinc. This might be given more weight were it not for the fact that the bronzes with zinc were cast, machined and tested some months after those without zinc. The "wearing-in" period in tests made in the presence of oil is known to be dependent quite largely upon the smoothness of the original contact surfaces, the oil, the metals themselves and other factors including contact pressures and rates of slip. Since the bronzes with and without zinc were not all prepared and tested at one time, the described differences in performance may not be the result solely of the additions of zinc.

But even if the results of the wear tests with oil are assumed to establish that zinc tends to increase the weight losses and duration of the "wearing-in" period, zinc will not necessarily exert a deleterious effect upon bronzes used in bearings since the importance of the "wearing-in" period is a function of the fit. The better the fit, that is, the smoother the original surfaces and the better the conformity of bearing and shaft, the less will be the practical importance of the "wearing-in" period and the effects of the zinc.

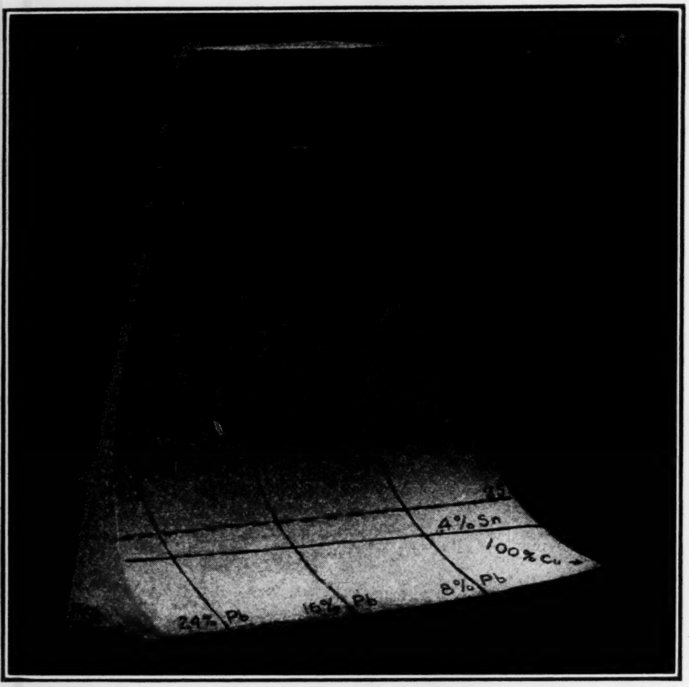


FIG. 15.—Resistance to Repeated Blows in Compression of Bronzes Tested at Atmospheric Temperatures.

Comparisons are based on the number of blows producing 5 per cent deformation.

With the possible exception of variations in the "wearing-in" period, the wear tests made in the presence of oil developed no deleterious effects from the addition of 4 per cent of zinc to the copper-tin-lead alloys. In this respect the tests with oil gave similar results to the majority of those tests made without oil which will be described subsequently in this report.

Trends in the Copper-Tin-Lead System:

Figure 12 shows that the rates of wear of the bronzes, in the absence of oil, were lowered by increase in either tin or in lead. However, the alloys with less than about 4 per cent of tin are not well suited for general bearing service since their wear rates remained relatively high even in the presence of appreciable proportions of lead. As is shown in Figs. 13 and 14(*a*), alloys with less than about 4 per cent of tin also showed a strong tendency toward "flat wheel" effects which may be taken to indicate low resistance to deformation. This is confirmed in Fig. 15, in which are summarized the results of the repeated pounding tests.

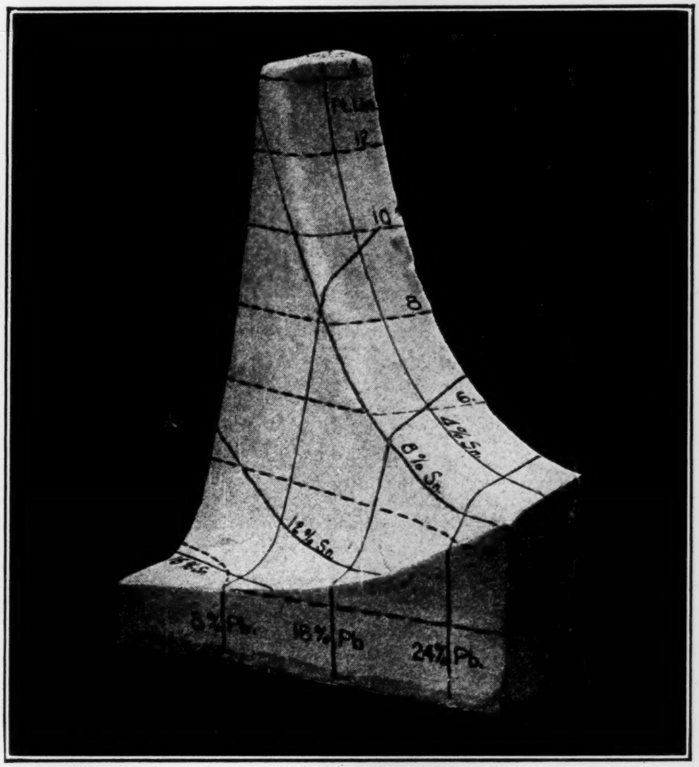


FIG. 16.—Izod Impact Resistance of Bronzes Tested at Atmospheric Temperatures.

Thus, a two-fold advantage was gained from the addition of the tin to copper in that the rate of wear was reduced and the strength or resistance to deformation was increased. On the other hand the copper-tin alloys, low in lead, had relatively unfavorable frictional properties, as is shown by the high torque values (Fig. 13), and also acquired rough worn surfaces (Fig. 14(*b*)). The worn surfaces of the alloys high in lead were relatively smooth (Fig. 14(*c*)) and the friction was relatively low. Both of these latter characteristics are favorable in preventing damage during any temporary inadequacy in lubrication.

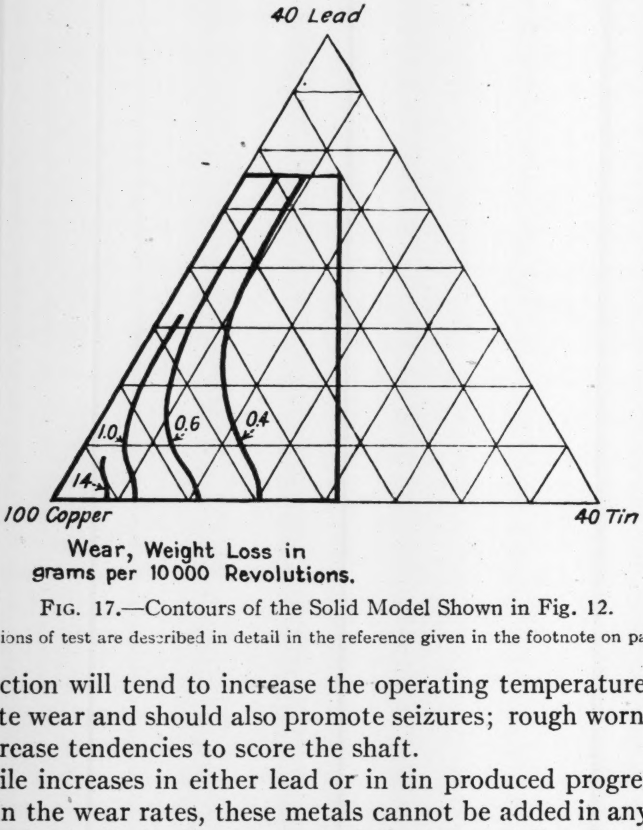


FIG. 17.—Contours of the Solid Model Shown in Fig. 12.

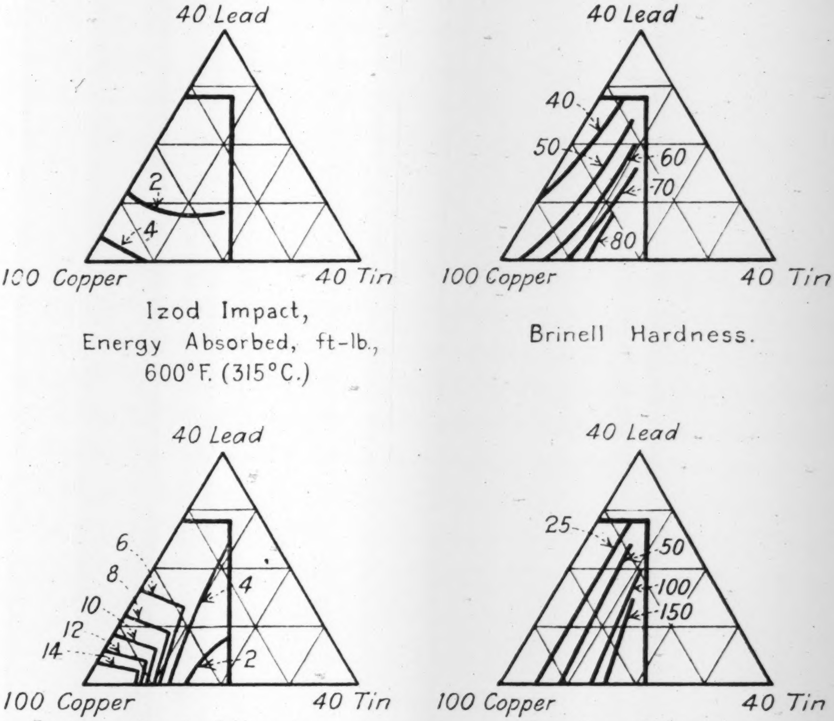
Conditions of test are described in detail in the reference given in the footnote on page 450.

High friction will tend to increase the operating temperatures and to accelerate wear and should also promote seizures; rough worn surfaces may increase tendencies to score the shaft.

While increases in either lead or in tin produced progressive decreases in the wear rates, these metals cannot be added in any proportions within the limits investigated without materially affecting other properties of interest in the application of bronzes in bearings. Figure 15 shows that increase in lead had only a minor effect upon the resistance to deformation under repeated blows in compression while tin markedly increased it. However, with more than about 7 or 8 per cent of tin, which resulted in the appearance of the alpha-delta eutectoid, there was a general decrease in the notch toughness (Fig. 16).

In alloys within the solid solution range, increase in lead also decreased the notch toughness appreciably, but in the high tin alloys the tin effectively reduced the impact values so that the effect of the increase in lead was not so noticeable.

Practical Interpretation of the Results for the Copper-Tin-Lead Alloys: Whether or not one composition may be considered to be superior to another is largely a question of the properties necessary to defeat the destructive forces encountered. Figures 17 and 18 show the contour lines of the solid models of Figs. 7, 12, 15 and 16, and are included to permit comparisons of the numerical values obtained in the tests of the different alloys.



Izod Impact, Energy Absorbed, ft-lb., 600°F. (315°C.)

Brinell Hardness.

Izod Impact, Energy Absorbed, ft-lb., Room Temperature.

Number of Blows to Produce 5 per cent Deformation.

FIG. 18.—Contours of the Solid Models Shown in Figs. 7, 15 and 16.

From the viewpoint of practical bearing applications, the copper-tin-lead alloys may be divided roughly into five groups, as is shown in Fig. 19.

Group A.—The first group, comprising alloys with less than about 4 per cent of tin, is unsuited for general bearing service since the alloys had very low resistance to deformation and wore rapidly in the absence of lubrication. Other reasons are shown in the various charts in this report. However, some of these alloys, such as those high in lead, may serve satisfactorily for service involving low loads.

Group B.—The second group, comprising alloys with less than about 5 per cent of lead and up to about 11 per cent of tin appears to be suited for ordinary conditions of service where neither very high strength nor the ability to operate in the absence of lubrication is required. The alloys were relatively tough, as judged by the Izod impact test, and those with the highest proportions of tin had moderately high resistance to deformation (strength).

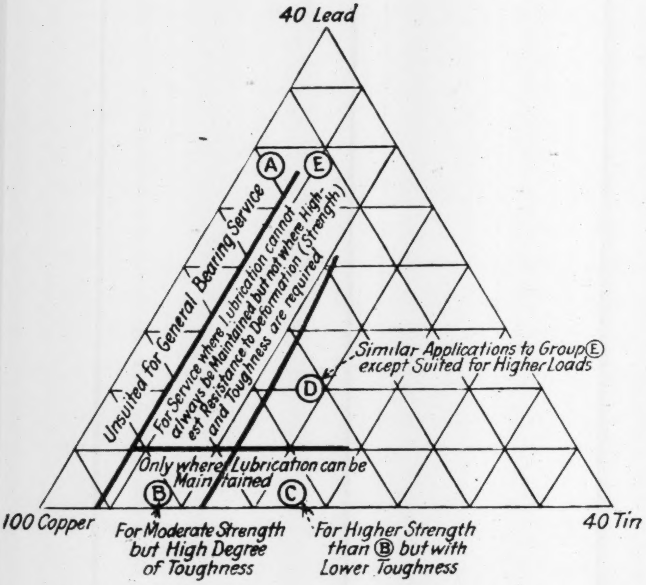

40 Lead
Ⓐ
Ⓔ
Unsuited for General Bearing Service
For Service where Lubrication cannot always be Maintained but not where Highest Resistance to Deformation (Strength) and Toughness are required
Similar Applications to Group Ⓔ except Suited for Higher Loads
Ⓓ
Only where Lubrication can be Maintained
Ⓑ
Ⓒ
100 Copper
For Moderate Strength but High Degree of Toughness
For Higher Strength than Ⓑ but with Lower Toughness
40 Tin

Fig. 19.—Grouping of Bronzes for Various Classes of Bearing Service.

Group C.—Alloys in Group *C*, containing from about 11 to 15 per cent of tin and less than about 5 per cent of lead were stronger but had a lower notch toughness than alloys of Group *B*. They should serve well where resistance to deformation is required but would not be chosen for applications when lubrication cannot be maintained.

Failures are more liable to occur at sharp oil grooves under suddenly applied loads than in the case of Group *B* alloys, especially those with the lowest proportions of tin.

Groups D and E.—Alloys in Groups *D* and *E* are best able of any of the alloys tested to operate for short periods in the absence of lubrication. They contain from about 4 to 16 per cent of tin and from 5 to 30 per cent of lead. As a group, the alloys did not have a high degree

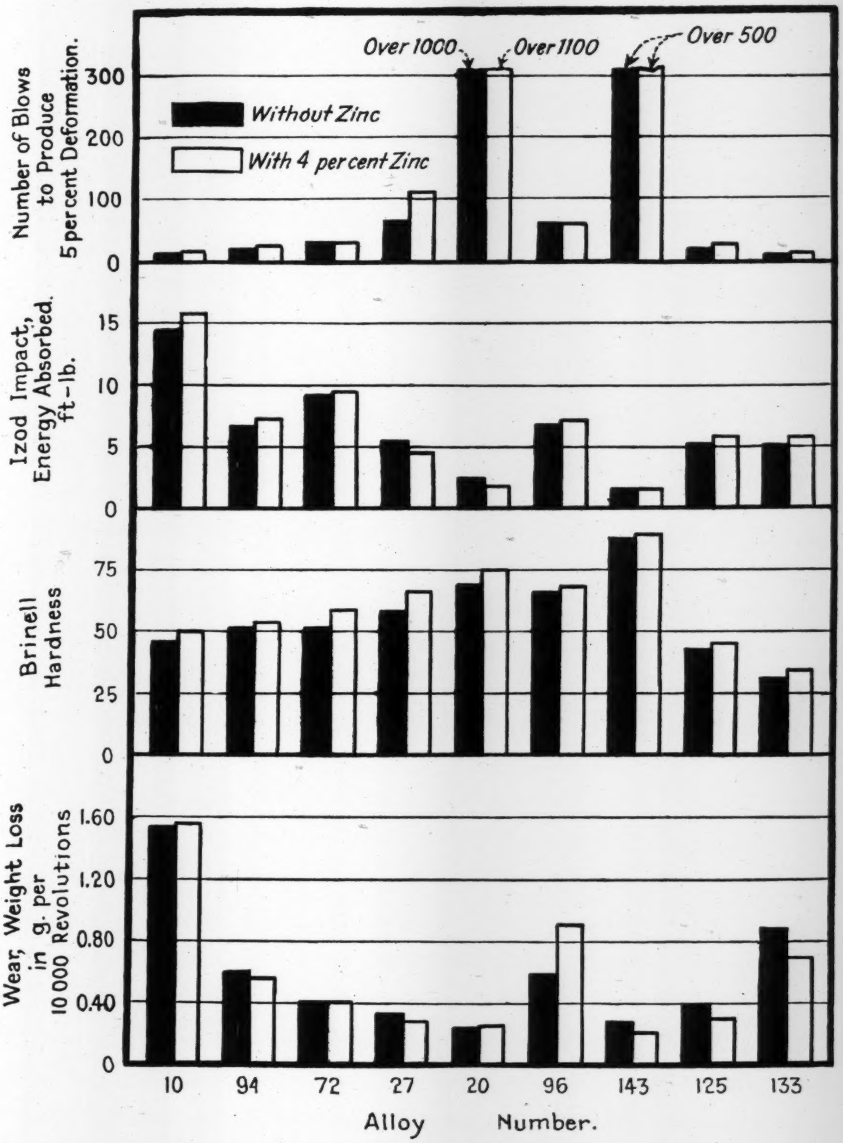


FIG. 20.—Comparison of the Properties of Bronzes With and Without Zinc When Tested at Atmospheric Temperatures.

For the chemical compositions of the various bronzes, see Table II.

of notch toughness but they had good resistance to deformation which increased with the proportion of tin. The alloys in these two groups had favorable frictional properties (low torque and smooth worn surfaces) especially when the lead was at the upper limits of the range. High lead tended to reduce the strength and notch toughness but appreciable variations can be secured in the mechanical properties of the group while maintaining a desirable set of wearing properties.

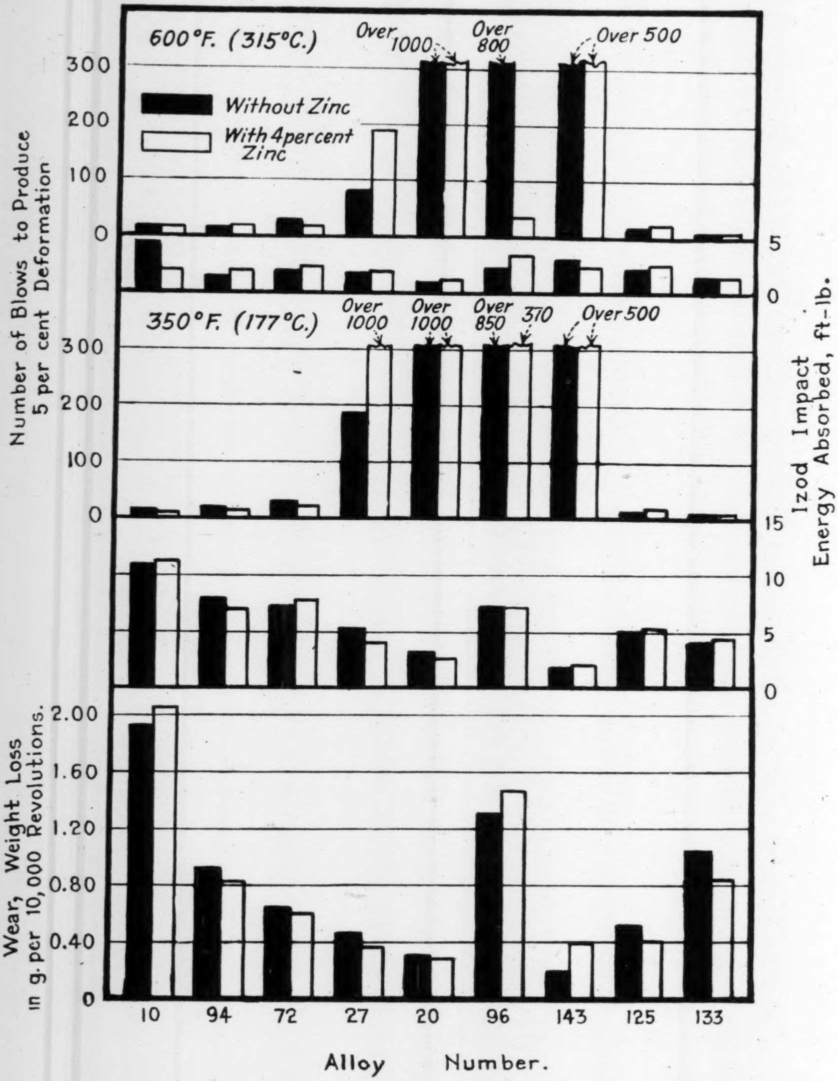


FIG. 21.—Comparison of the Properties of Bronzes With and Without Zinc When Tested at 350 and 600° F. (175 and 315° C.).

For the chemical compositions of the various bronzes, see Table II.

Effect of Temperature on the Properties of the Copper-Tin-Lead Alloys:

The data given in support of the preceding discussion were obtained in tests at atmospheric temperatures, but the grouping of the alloys and the general conclusions relating to their fields of application apply, with few exceptions, at elevated temperatures. The results of the tests at elevated temperatures differed numerically from those at atmospheric temperatures, but the trends in the copper-tin-lead system were generally similar. Increase in temperature raised the wear rates and lowered the notch toughness, but the pounding resistance was not appreciably affected within the range 70 to 600° F. (20 to 315° C.). These features can be substantiated by comparison of Figs. 20 and 21.

An important effect of temperature increase was to reduce the notch toughness of all the alloys to very low values. In other words the differences in Izod impact values between the different alloys at atmospheric temperatures disappeared almost entirely when the temperature was raised to 600° F. (315° C.) (Fig. 18).

Effect of 4 per cent of Zinc on the Properties of the Copper-Tin-Lead Alloys:

Zinc, when added in small amounts to copper-tin alloys, acts as a deoxidizer and the resultant castings are generally freer from oxides and blowholes than untreated metal. When present in quantities greater than 2 or 3 per cent, zinc is generally considered to have a deleterious effect on the strength and ductility of bronzes. However, copper-tin-zinc alloys are widely used industrially and some have very excellent properties. The work of Hoyt[1], Guillet and Revillon[2] and Thurston[3] gives a fairly complete picture of the constitution and properties of alloys in this system but much less is known of the effect of zinc on the copper-tin-lead alloys so widely used as bearing metals.

Clamer[4] found that the addition of zinc to leaded bronzes hardened the alloys considerably, embrittled them and increased the wear, but he concluded that the alloys having approximately 5 per cent of tin, up to 20 per cent of lead and up to 5 per cent of zinc should be entirely satisfactory for all classes of car journal bearings.

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[1] S. L. Hoyt "On the Copper-Rich Kalchoids," *Journal*, Inst. Metals (British), Vol. 10, p. 235 (1913).

[2] L. Guillet and L. Revillon, "Determination of the Coefficient of Equivalence for Special Bronzes," *Revue de Metallurgie Memoirs*, Vol. 7, p. 429 (1910).

[3] R. H. Thurston, "Materials of Engineering," Part III, Brasses, Bronzes (1900), John Wiley and Sons, p. 172.

[4] G. H. Clamer, "Effect of Changes in the Composition of Alloys Used by the American Railways for Car Journal Bearings," *Transactions*, Am. Inst. Metals, Vol. 9, p. 241 (1915).

In studying the copper corner of the copper-tin-zinc system Kuhnel and co-workers[1] found that a copper-base alloy containing 9 per cent of zinc and 6 per cent of tin was more resistant to wear than one containing 8 per cent of tin and 3.5 per cent[2] of zinc.

The addition of 4 per cent of zinc to the copper-tin and copper-tin-lead alloys previously discussed in this report resulted in the following effects which are shown graphically in Figs. 20 and 21:

Hardness.—A measurable but slight increase was produced in the Brinell hardness.

Structure.—The zinc increased the amount of the eutectoid as will be seen in Figs. 9(*a*) to (*f*) inclusive, and in this respect may be considered to replace tin although not in equal proportions. Zinc also appeared to promote uniformity in the distribution of the lead with respect to both size and location of the particles, but this was evident only in the low-lead bronzes, Figs. 8(*c*) and (*d*), and not in the high-lead bronzes, Figs. 8(*e*) and (*f*). Further work would be necessary to establish the magnitude of the effects in this direction.

Impact.—Comparatively minor changes were produced in the Izod impact test values. In some cases the alloys with zinc showed slightly higher impact resistance and in others lower values than the corresponding zinc-free alloys but the differences were small at all temperatures between 70 and 600° F. (315° C.) and should not affect the selection of any one of the metals for bearing service.

Repeated Pounding.—Small changes were observed in the resistance to repeated blows in compression but the tendency was generally toward an increase in the resistance to deformation at each test temperature including 600° F. (315° C.).

In Fig. 21 there is a large difference in the numerical values used in comparing alloy No. 96 with and without zinc at 600° F. (315° C.). This difference is not indicative of a large difference in the resistance to deformation but is the result of the method of comparison employed, which is based on the number of blows producing 5 per cent deformation of the test specimens.

It so happened that the bronze with zinc deformed about 6 per cent and that without zinc only about 3½ per cent in the first fifty blows. Both then showed evidence of strain hardening with the result that the bronze without zinc required a very large number of blows to produce 5 per cent deformation. Actually the deformational characteristics of the alloy No. 96 were quite similar with and without

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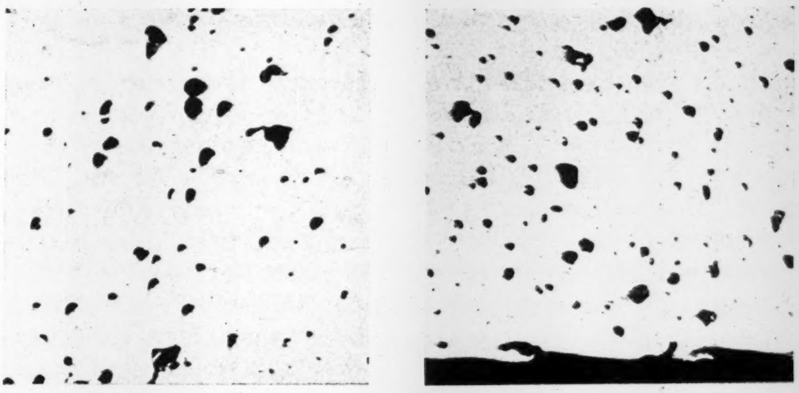
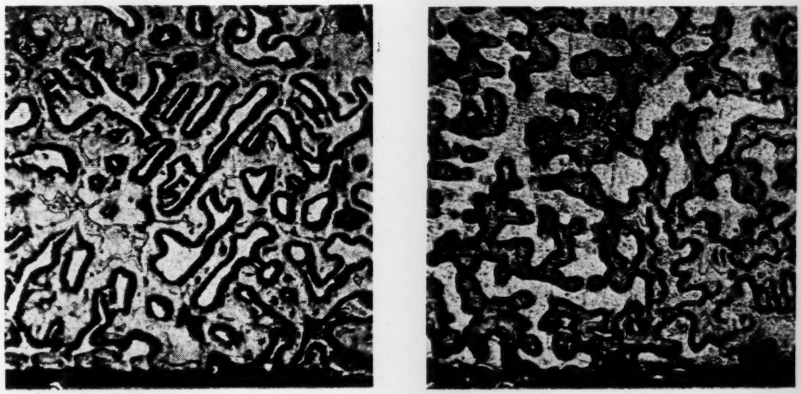
[1] R. Kuhnel, "The Constitution and Properties of Red Brass," *Zeitschrift für Metallkunde*, Vol. 18, p. 306 (1926); *Journal*, Inst. Metals (British), No. 1, p. 465 (1927).

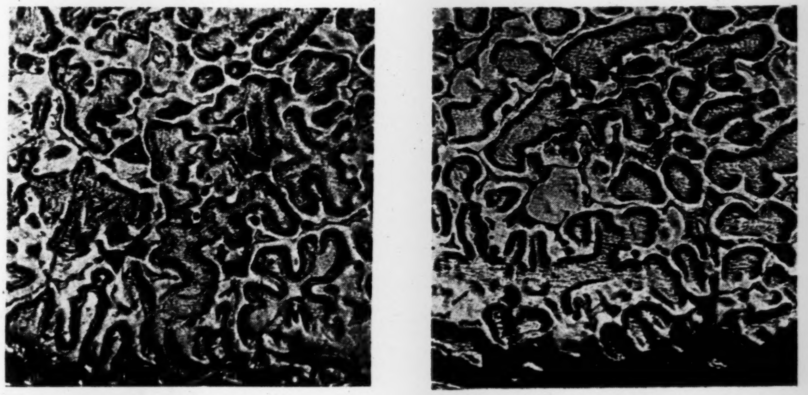
[2] Both bronzes contained 2 per cent lead and small amounts of nickel and iron.



(*a*) From first two melts, without zinc. (*b*) From first two melts, with 4 per cent zinc.



(*c*) From first two melts, without zinc, etched. (*d*) From first two melts, with 4 per cent zinc, etched.



(*e*) From third and fourth melts, without zinc, etched. (*f*) From third and fourth melts, with 4 per cent zinc, etched.

FIG. 22.—Micrographs of Different Melts of Alloy No. 96 With and Without Zinc (× 100).

In the case of etched specimens the solution consisted of 3 parts NH_4OH plus 1 part H_2O_2 followed by solution of $FeCl_3$ in HCl (10 g. $FeCl_3$ plus 30 cc. concentrated HCl plus 120 cc. H_2O).

zinc, as is shown by the fact that at the end of 1000 blows the bronze without zinc had deformed 5 per cent and that with zinc 7.5 per cent.

Wear and Friction.—The zinc quite generally lowered the friction in the wear tests made without oil, as is indicated by the work values recorded in Table V, but seemed to exert an irregular influence upon the wear rates. In a majority of cases the weight losses were not appreciably different in the corresponding alloys with and without the zinc, but in alloys Nos. 125 and 133 containing high lead and low tin (2 to 4.5 per cent of tin) the zinc produced a definite decrease in the weight losses. On the other hand in the high-tin low-lead bronzes, Nos. 96 and 143, the addition of 4 per cent of zinc increased the wear rates at one or both test temperatures. The increase was especially marked in alloy No. 96 containing 88 per cent of copper, 10 per cent of tin and 2 per cent of lead although the concordance of duplicate determinations was not so good as with most of the other alloys.

TABLE V.—FRICTIONAL PROPERTIES OF BRONZES WITH AND WITHOUT 4 PER CENT OF ZINC AS DETERMINED IN THE WEAR TESTS WITHOUT LUBRICATION AT DIFFERENT TEMPERATURES.

Alloy[a]	Total Work, m-kg., Done at the End of the First 40,000 Revolutions			
	Tests at Room Temperature		Tests at 350° F. (175° C.)	
	Alloy Without Zinc	Alloy with 4 per cent of Zinc	Alloy Without Zinc	Alloy with 4 per cent of Zinc
No. 10	26 500	24 800	17 500[b]	15 000[b]
No. 94	22 300	21 800	20 500	16 000
No. 72	21 500	21 000	18 500	17 200
No. 27	24 100	20 100	17 000	16 000
No. 20	23 200	21 800	19 000	19 000
No. 96	29 800	26 500	21 000	20 000
No. 143	46 600	41 800	40 400	36 200
No. 125	18 900	19 500	14 000	12 200
No. 133	15 000	14 300	14 500	14 500

[a] See Table II for chemical compositions.

[b] For first 30,000 revolutions.

A second lot of castings was made of alloy No. 96 and, as in the first group, zinc increased the rate of wear. Microscopic examination showed no features to which the reversal in the effects of zinc could justly be ascribed. As is shown in Figs. 22(*a*) and (*b*), a reasonably uniform distribution of the lead particles was found, both in the alloys with and without zinc, and the structures in the two lots of each alloy were quite similar (Figs. 22(*c*) to (*f*) inclusive). It may be significant that while more of the eutectoid was shown by the alloy with zinc in the second group of castings the opposite was true in the first group (Figs. 22(*c*) and (*d*)). This does not explain the differences in wearing properties since in both groups the alloy with zinc showed the higher wear. However, it does indicate that the alloy under discussion is sensitive to casting conditions and examination of Fig. 12 will show that this composition is also in a field of rapidly changing wear rates.

Karr[1] showed that variations in the casting conditions might have an appreciable effect upon the properties of bronzes. Similarly, the fact that very marked changes in properties can be encountered as the result of relatively small differences in a variable such as casting temperature is shown by the results reported more recently by Rowe[2] for an alloy containing 88 per cent of copper, 6 per cent of tin and 6 per cent of zinc.

When these effects are viewed in the light of the generally small changes produced by zinc over the greater portion of the field covered in the system copper-tin-lead, it would seem justifiable to draw the conclusion that the effects of zinc up to 4 per cent are small and may be insignificant in comparison with changes in properties quite readily produced from variations in the casting conditions.

While the foregoing comparisons of the wearing properties of the bronzes in the absence of lubrication are based on the weight losses per 10,000 revolutions, similar effects are shown when comparisons are based on the wear per unit of work. It was, therefore, unnecessary to include both sets of comparisons in this report.

It should also be noted that no measurable wear was produced on the steel specimens in any of the wear tests at atmospheric temperatures or at 350° F. (175° C.). In most cases the steel specimens gained a little in weight through the adherence of bronze particles.

SUMMARY

Based on hardness tests, Izod impact tests, repeated pounding tests and wear tests, both with and without lubrication, at temperatures within the range 70 to 600° F. (20 to 315° C.), the bronzes in the copper corner of the copper-tin-lead system have been classified according to the character of service for which they seem to be best adapted.

Bronzes with less than about 4 per cent of tin are unsuited for general bearing service since they had low resistance to deformation and wore rapidly in the absence of lubrication. However, some of these alloys, such as those high in lead, can serve satisfactorily for special service involving low loads.

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[1] C. P. Karr, "Standard Test Specimens of Zinc Bronze," U. S. Bureau of Standards *Technologic Paper No. 59* (1916).

[2] F. W. Rowe, "The Effect of Casting Temperature on the Physical Properties of a Sand-Cast Zinc-Bronze," *Journal*, Inst. Metals (British), Vol. 31, p. 217 (1924).

Bronzes with less than about 5 per cent of lead appear to be suited only for service where lubrication can be maintained. However, they are applicable with such a restriction to a wide range of service conditions depending upon the proportions of tin present. With low tin the alloys were tough but did not resist deformation as well as the bronzes high in tin.

Bronzes containing more than about 5 per cent of lead are best able, of any of the groups studied, to operate for short periods in the absence of lubrication. Bronzes with 15 per cent of lead are better in this respect than bronzes with 5 per cent of lead but there were no appreciable advantages apparent in raising the lead above about 15 per cent. In fact some disadvantages were encountered in that the toughness was decreased in bronzes containing around 4 to 8 per cent of tin. However, by varying the proportions of tin and lead in the bronzes of this group, varied mechanical properties can be obtained while maintaining good frictional properties.

The addition of 4 per cent of zinc to the bronzes had, in general, small influence upon the properties of the bronzes studied. With two exceptions such changes as were observed seemed beneficial rather than detrimental for bearing service since they comprised a tendency toward higher hardness and resistance to deformation, lower friction and lower wear in the absence of oil.

Further development of methods of test for wear in the presence of lubricants may show that zinc tends to increase the weight losses and duration of the "wearing-in" period of bronze bearings, but since this is also a function of the perfection of fit such effects would be disadvantageous only in certain cases. The addition of 4 per cent of zinc did not appreciably affect the wear rates subsequent to the "wearing-in" period.

In general, the results seem to justify the conclusion that the effects of zinc up to 4 per cent are generally small and may be insignificant in comparison with changes in properties quite readily produced in bronzes from variations in foundry practice. This should not be construed to apply to additions of zinc when other impurities are present.

Acknowledgments:

Acknowledgment is made to H. K. Herschman, Associate Metallurgist, for the micrographs and to E. R. Darby, Metallurgist, The Bunting Brass and Bronze Co., for his cooperation throughout all phases of the work.

DISCUSSION

MR. R. J. SHOEMAKER[1] (*presented in written form*).—It is very interesting to note that 4 per cent of zinc had little or no effect on the anti-frictional quality of bearing bronze. This has been borne out in tests of leaded bronze locomotive bearings in actual service. The Magnus Co. has made a number of tests in the past on locomotive bearings containing a considerable quantity of zinc and these castings wore just as well as those which contained little or no zinc. In fact the presence of zinc seemed to be beneficial rather than harmful.

Zinc should be classed as a desired element rather than as an impurity in bearing bronze, as this element has a decided strengthening and toughening effect and increases the life of the castings when they are subjected to impact stresses, without decreasing the anti-frictional value of the metal.

It may be of interest to note that locomotive driving brasses are commonly fastened to the steel housing by means of brass pins driven through the crown. These pins are usually of steam metal composition containing 85 per cent of copper, 5 per cent of tin, 5 per cent of zinc and 5 per cent of lead. In so far as is known, the contact of this metal with the journal produces no bad effects in service.

With regard to the data presented by Messrs. French and Staples on the rate of wear of bearing bronze containing low lead, their results have been borne out in a test made of sand-cast floating rod bushings on one of the leading railroads a year or two ago. In this test, a standard mixture containing about 8 per cent of tin and 12 per cent of lead was compared with one containing 10 per cent of tin and 5 per cent of lead. The mixture which had the higher lead showed less wear in service than the low lead and high tin mixture.

As pointed out by Messrs. French and Staples, so long as perfect lubrication can be maintained the composition of the bearings is secondary, and it is only when the lubrication breaks down or is incomplete throughout the surfaces in contact with the journal that the question of composition is of prime importance.

One of the most important points in connection with the wear on bearing metal is the running temperature. The lower this temperature, the less wear there will be on the bearing.

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[1] Magnus Co., Inc., Chicago, Ill.

It is interesting to note that on European railway locomotives practically all of the bearings are lined with white metal. They have found that with the use of white metal much better results are obtained than with bronze, due to the lower running temperature of the bearing. On these locomotives, oil lubrication is used throughout with fewer failures from hot boxes, etc., than was formerly the case with grease lubrication in contact with bronze.

I wish to congratulate Messrs. French and Staples on their excellent paper as it is a valuable contribution to the science and technique of wear testing of metals.

MR. H. E. SMITH.[1]—In the practical manufacture of railroad bearings, so far as I know, zinc is never intentionally added. When we find it we consider it an index of the use of common yellow brass (copper and zinc) scrap in the metal charge. It is quite possible that in such metal the effect of zinc, or, rather, the effect of the addition of scrap as indicated by the zinc, is more pronounced than if new clean zinc were used.

MR. W. M. CORSE.[2]—I think the work as described by Messrs. French and Staples reflects the splendid cooperation that this subject has received from a number of members of the staff of the Bureau of Standards. As a result of the paper that was presented last year[3] the specifications for railroad bearings I think have been materially reduced in numbers.

If I understood correctly, this work, which has more reference to the automobile bearing industry than it does to the railroad bearing industry, may affect the same saving in the number of types of bearings with which we are now familiar and with which we have to contend in that industry. If that reduction in the number of types of bearings comes about as a result of this work at the Bureau it will be quite a contribution to the economics of the situation.

Something that has not been stressed particularly by Messrs. French and Staples, but which always comes up in a discussion on this sort of work, is the value of unlubricated tests of this kind. At the Bureau a piece of apparatus has been devised for developing the effect of metal to metal contacts in bearings, and a good many of us who have been in touch with this work for several years feel that wear only starts where one has metal to metal contact and, therefore, study of dry surfaces in bearing testing does give valuable information. I doubt whether anyone would recommend having no lubrication, because, as pointed out in the paper, lubrication is a very important factor in this whole situation, but if we can leave the variables out, such as lubrication, and study one phase of it we may get some information on what has always been a very complex subject, at least so far as the testing of the materials is concerned.

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[1] Engineer of Materials, New York Central Lines, New York City.

[2] Metallurgical Engineer, Washington, D. C.

[3] H. J. French, "Wear and Mechanical Tests of Some Railroad Bearing Bronzes," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 298 (1928).

MR. ALVAN L. DAVIS.[1]—In practice bronze bearings wear against some other metal, generally steel, either hardened or unhardened, or against cast iron. It would add considerable further interest if the Bureau were to develop figures showing the relative life of the different bronzes against cast iron, and steel, both hardened and unhardened.

MR. H. J. FRENCH[2] (*authors' closure*).—The only comment requiring reply is, I believe, that made by Mr. Davis, who is giving a large order when he suggests that a study be made of the effects of different ferrous metals on the wear of bronzes or other non-ferrous metals. However, I can say at this time, that there have been some tests made with both hardened and unhardened steels running in contact with some of the copper-tin-lead alloys. I am not in a position to give you the exact wear rate values at this time, but I can say that in our preliminary tests the contact pressure appeared to be more important than a change from unhardened to hardened or tempered steels of similar compositions.

Care must be used in interpreting this statement because such materials as nitrided chromium-molybdenum-aluminum steels, in all probability, have very different properties than ordinary steels with respect to surface adhesion phenomena. Therefore, the foregoing comment may apply only to a restricted group of metals or set of conditions. For example, it is already known that non-ferrous metals which will not last long in contact with ordinary steels, in the absence of lubrication, may run for a considerable period of time when in contact with something like chromium plate or the nitride-hardened steels.

Contact pressure variations not only change the wear rates directly but may have a large indirect influence through variations in tendencies toward film formations. Such films were discussed in detail in a previous paper[3] and were found to be most readily produced in copper-tin-lead alloys high in lead and in sand castings. It has been found that lowering the hardness of the steel with which the bronze is in contact seems to promote the formation and maintenance of these films, which very often act as protective coatings to reduce the wear.

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[1] Research Metallurgist and Engineer, Scovill Manufacturing Co., Waterbury, Conn.

[2] Senior Metallurgist, U. S. Bureau of Standards, Washington, D. C.

[3] H. J. French, "Wear and Mechanical Tests of Some Railroad Bearing Bronzes," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 298 (1928).

Factors such as these must be considered in attempting to answer an inquiry as to the effect of the materials in contact with the bronzes, and I cite them to illustrate the complexity of the subject and the difficulty of giving an adequate answer to Mr. Davis' question at this time.

ALUMINUM ALLOYS FOR PRESSURE DIE CASTINGS

By Sam Tour[1]

Synopsis

The properties required in finished aluminum-alloy pressure die castings are enumerated and briefly described. The properties required of aluminum alloys to make them suitable for the pressure die-casting process are enumerated and described. The effect of the various steps of the process on these properties are discussed as well as the relation between alloy specifications and the effect of impurities.

The effects of such impurities as iron, zinc, manganese, magnesium, silicon, tin, etc., on aluminum-copper, aluminum-copper-silicon, aluminum-silicon aluminum-copper-nickel-silicon and aluminum-silicon-tin alloys are discussed and a summary presented in the form of a table showing how these various impurities sometimes impair and sometimes improve such properties as fluidity, shrinkage, solvent action on iron, hot shortness, strength at elevated temperatures, machineability, polishing properties and plating properties. The iron content problem is briefly pictured, as is also the question of hard spots.

The great need for systematic and scientific study of the effect of various impurities on the casting properties and final properties of the different aluminum-base alloys used in the pressure die-casting industry is stressed. Curves showing the results of such a study made in Germany on sand-cast 13-per-cent silicon-aluminum alloy are given as an example of how each of the pressure die-casting alloys should be studied. It is recommended, however, that these studies be made on pressure die-cast specimens and not on sand castings and that they be cooperative tests carried out through some such procedure as is being followed now in the work of the Sub-Committee on Die-Cast Metals and Alloys of the Society's Committee B-2 on Non-Ferrous Metals and Alloys, in order that the final results may be authentic and authoritative.

"Pressure die casting" is defined as the making of castings by the forcing of molten metal under pressure into metallic molds or dies. This article discusses the requirements which aluminum-base alloys must meet in order to be suitable in the pressure die-casting process and the effect of various impurities in the alloys.

Requirements

The consumer's requirements may be briefly classified under the following headings:

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1. Strength,
2. Hardness,
3. Ductility,
4. Resistance to attack,
5. Permanency,
6. Machineability,
7. Adaptability to special finishes,
8. Polishing properties,
9. Appearance,
10. Weight, and
11. Cost.

A study of these requirements in a specific case usually narrows the field of alloys available to the producer to a very few alloys with which he has had experience. The alloys must also be considered by the producer from the viewpoint of requirements imposed by the pressure die-casting process, his particular type of equipment, and his experience.

The producer's alloy requirements may be briefly classified under the following headings:

1. Adaptability to the type of machine available,
2. Melting point,
3. Fluidity,
4. Range of solidification,
5. Strength at elevated temperature,
6. Ductility at elevated temperature,
7. Solidification shrinkage,
8. Cooling shrinkage,
9. Rate of attack on iron containers,
10. Rate of attack on die materials,
11. Sensitivity to impurities, and
12. Limits of impurities.

All of these may of course be summed up under the one term, "pressure die castability." In general it is this summation that the producer considers, and he of necessity must consider it in the light of his own past experience with the different alloys. In a field of metallurgy where so little information is given out as in the pressure die-casting alloy field, there has been very little. if any, opportunity for one producer to discuss experiences with another producer. The result has been that each producer has arrived at his own viewpoint of the alloys and has developed his practice and his equipment in line with this viewpoint. A secondary result of this has been that for a given set of consumer's requirements, one producer finds one alloy most suitable while a second producer finds a different alloy most suitable.

A.S.T.M. DIE-CASTING ALLOY COMMITTEE

It is this situation primarily that was responsible for the formation of Sub-Committee XV on Die-Cast Metals and Alloys of the

Society's Committee B-2 on Non-Ferrous Metals and Alloys and in the organized program of tests of this sub-committee.[1] A brief survey of the field of effort of this sub-committee in connection with aluminum-base alloys shows that the committee is trying to determine the relative mechanical and chemical properties of test-bar castings made from the most commonly used pressure die-casting alloys. The program as laid out is a good one and a fair one and should throw considerable light on the subject.

Test specimens made by the different producers cooperating in the work of the sub-committee have been found to vary more or less from the nominal desired compositions. Differences in properties of the test specimens as determined by the sub-committee cannot be ascribed wholly to the variations from nominal compositions nor to the various impurities found on analysis. The unknown variables of the pressure die-casting process itself as practiced by the different producers may be largely accountable for the differences.

A program of tests to determine the effect of variations in analysis and the effect of impurities would be a laudable although a very large undertaking. It is a program to be looked forward to and it is hoped that it will become possible in the near future.

ALLOY COMPOSITION SPECIFICATIONS

Certain considerations should be borne in mind in formulating specifications for die-casting alloys. Specifications, for example, which limit zinc content in an aluminum-base alloy die casting to 0.75 per cent would indicate that a higher zinc content would adversely affect the desired properties in the finished casting. No authentic information to support this is available. Such requirements place undue burden on the producer without any advantage to the consumer. When a specification allows 0.3 per cent impurities other than iron and zinc in an alloy, it indicates that impurities in that amount will not adversely affect the properties of the finished casting. Yet we know that the properties of aluminum alloys are very greatly affected by small amounts of magnesium, manganese, etc. In this case the requirement places an undue burden on the consumer. Specifications which place limitations on either the producer or consumer without corresponding advantage to the other party to the contract are harmful.

REQUIREMENTS FOR PRODUCTION

In considering the required properties of pressure die-casting alloys from the production standpoint, let us start with virgin raw metals and follow them through to the finished casting. If iron pots are used for alloying, there will be a certain amount of iron picked up in this operation. The alloy must be such that this iron plus the iron which will be picked up later in the pressure die-casting process itself will not be detrimental. If silicon in the form of metallic silicon, nickel as nickel shot, copper as copper clippings, etc., are used, some over-heating of the metal in order to effect good solution is inevitable. Also some oxidation will take place and fluxing and skimming off of dross should be practiced. Fluxes employed on aluminum usually contain zinc chloride or magnesium chloride or both. Their use introduces into the aluminum certain amounts of zinc and magnesium. The alloy must be such that these additions will not be detrimental. When the alloy is ready for the pressure die-casting machines, about 0.3 per cent of iron has been picked up and a trace of either magnesium or zinc, also unknown amounts of gases and oxides.

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[1] See *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 202 (1928).

In die-casting machines, iron melting pots and iron pressure retorts are used and there is the inevitable iron pick-up from them. The alloy must be such that it does not too readily attack these machine parts and also such that the iron picked up here will not be detrimental. On the average about 0.5 per cent of iron is picked up here from the time the metal is put in the machine until it is made into a casting. If the alloy is one which requires a high casting temperature or if the design of the casting and the die is such as to require a high casting temperature, the rate of attack on machine parts will be greater and the iron pick-up will be greater. In addition to this iron pick-up, the metal has also taken up an unknown amount of gases and oxides.

Air pressure suddenly applied forces the metal to enter the die, filling the gate, runners, casting cavity, vent and over-flow cavities. The entire operation of filling is accomplished possibly in a fraction of a second, the molten metal traveling at very high velocities through the constricted inlet and through the thin sections of the casting. Undoubtedly when the metal begins to enter the cavity and before the cavity is filled, there is considerable breaking up of the metal stream into individual globules or drops of molten metal. The metal must be such that the individual particles will join together again to form a continuous unbroken casting free or reasonably free from run marks and other surface imperfections.

Surface Finish:

The venting of the air in the die as the metal rushes in can usually be so arranged as to avoid large air pockets, and inlets can be so distributed as to eliminate swirls to a considerable extent, but even under these improved conditions some alloys will give much better surface finish than others. Where surface finish is of considerable importance, as is the case with a very large percentage of the output of the pressure die-casting industry, it is often necessary to change from one alloy to another in order to obtain the desired result. Often it is impossible to predict from a study of the casting itself and of the arrangement of the inlets, vents, etc., which alloy will give the best surface appearance. The best alloy must be found by trial, choosing of course those alloys which will give castings of the required properties.

It is often surprising what great effects are produced by small variations in alloys and by small amounts of impurities. In many cases 0.05 to 0.10 per cent of magnesium will make good surface finish impossible. Iron content below 1.25 or 1.50 per cent sometimes makes good finish impossible. Iron abnormally high often has a similar effect. Oxides in solution and gases in solution no doubt have important effects, but the necessary methods of analysis have not been available to permit of any statements concerning their effects.

Erosion of Dies:

The molten metal impinges on various portions of the die, sometimes directly and sometimes obliquely, and the metal must be such that it will not unduly attack and erode the die materials. Graphite, oils, clay washes, mica preparations, etc., are used to coat the dies periodically to assist in withstanding this attack. Great differences are found, however, in the alloys in this respect. Temperature, both of incoming metal and of the die proper, is of course of fundamental importance. Size of inlets with respect to weight of metal passing are also of importance. In addition, however, the properties of the alloy are important. Here we find that iron has very great effect. Alloys low in iron sometimes will ruin the most expensive die in a few hours, while alloys with iron at 1.5 to 2.0 per cent will give no trouble at all. Some high-silicon alloys seem to have a much greater tendency to attack the dies than other alloys lower in silicon. Although iron might correct this trouble with high-silicon alloys, its presence cannot be allowed in a sufficient amount as it ruins the mechanical properties of the alloy in the finished casting.

Fluidity of Molten Metal:

The instant the metal enters the die it begins to cool. The alloy must have sufficient fluidity and range of fluidity and semi-fluidity or plasticity between allowable maximum casting temperature, as limited by its effect on machine parts and die materials, and final solidification to allow of complete filling of the die cavity. What affects this fluidity? In some aluminum alloys, particularly the copper-aluminum and silicon-aluminum alloys, iron to a limited percentage improves flowing properties. The higher the copper content, the less is the effect of iron with regard to this property. In the silicon alloys the iron, however, is more apt to act to the detriment of the mechanical properties in the final casting. Sometimes foreign matter, such as dissolved gases, suspended oxides, etc., cuts down fluidity, and a good fluxing with zinc chloride or magnesium chloride is helpful in restoring it. But here again a new trace of zinc or magnesium is added to the metal.

Shrinkage, Porosity and Blowholes:

When the metal solidifies in the die it does so from the surface inward to the center of the sections as cast, during which the natural solidification shrinkage takes place. The surface is fed from the center of the section and natural solidification shrinkage therefore concentrates in the center of the sections and causes porosity. Feeding by means of risers as in sand-casting or permanent-mold casting practice is impossible. Porosity at least equal to the liquid shrinkage from casting temperature to solidification temperature plus solidification shrinkage must be present in pressure die castings. The alloy, then, should have as small a liquid shrinkage and as small a solidification shrinkage as possible. This raises the question as to which alloy additions to aluminum decrease and which increase solidification shrinkage. Very meager information is available in this field. We know that in making certain rich aluminum-alloy hardeners, rising of the ingot or exudation occurs. This is sometimes considered to indicate that the alloy expands on solidifying, but recently it has been shown that this phenomenon is quite closely associated with the amount of dissolved gases in the molten metal just before solidification and the ability of the metal to retain these gases on solidification. Nevertheless, in a given case it is often found that porosity in the center of a perfectly vented section varies with different alloys. It of course varies considerably with the degree of superheat of the metal. The higher the casting temperature above the final solidification temperature, the greater is the porosity. The alloy must therefore not only have minimum liquid and solidification shrinkage, but it must have fluidity at as low a superheat as possible. Indications are that silicon here is an important factor as is also iron, manganese and magnesium. Silicon seems to increase fluidity and decrease shrinkage provided iron is kept to a proper proportion. Iron in amounts increasing from 1.50 to 3.0 per cent in copper-aluminum alloys with copper decreasing from 14 to 4 per cent seems to decrease shrinkage to some extent. Manganese with medium low silicon and low iron seems to be beneficial, whereas with higher silicons it sometimes seems harmful.

Combined with the questions of shrinkage and porosity is the question of gases in solution in the molten metal and the ability of the alloy to retain these gases on solidification. Pressure die casting requires alloys which will not absorb undue amounts of gases while liquid and which have the property of retaining as much of the gases as possible on rapid solidification. The rapid chilling and solidification which takes place in pressure die casting assists in retaining the gases in solution. The rate of solidification of the metal in the die decreases from the surface to the center of the section as cast. Although the rate of solidification is high, there is nevertheless some segregation, and the last metal to solidify is more highly charged with gases than the first. Surplus gases which cannot be held in solution are therefore concentrated at the center of the section, causing additional porosity to that caused by shrinkage. These gases accumulate in the center of sections under considerable pressure, in some cases sufficient to cause a growth of the casting after it is removed from the die. If the gas accumulation is near the surface and the casting is removed from the die rather hot, the gases expand locally and cause blistering. Often blisters are due to air entrapped by the metal plugging the vents before the air is expelled.

Contraction and Hot Shortness:

After the metal has solidified in the die the properties that are important are coefficient of expansion or rather contraction, hot shortness, ductility and strength of the alloy. By contraction is meant the actual contraction that the alloy would have in cooling from the temperature of solidification down to the temperature of removal from the die, if the casting were allowed to cool unrestrained. In pressure die casting the steel dies do not allow for any actual contraction during this period, and therefore the solidified metal must stretch an amount equal to this contraction. The hotter the die and the sooner the casting is removed from the die after solidification, the less stretching will take place. The time interval between forcing in of metal and opening of die and removal of casting is a great variable. It cannot be too short as the metal must be completely solidified and must cool enough to develop sufficient mechanical strength to hold its shape before the die can be opened at all. If large sections are being cast around cores and slides, the metal must have sufficient strength to withstand the strains of removing these cores and slides. Aluminum alloys are very weak near the melting or solidification temperature, gaining strength as the temperature drops. The time interval between filling the die and removing the casting must be such as to allow the casting to cool somewhat and gain strength. Since while cooling it is stretching, hot shortness is a great problem, as it produces cracked castings. The strength and ductility of aluminum alloys at elevated temperatures are important considerations, as they determine the length of time the casting must be allowed to cool before removal from the die. The nature of the ductility, too, is important. The stretching during cooling should take place uniformly throughout the entire length of the section of casting held in the die. There should be no "necking down" or localized ductility. When the nature of the alloy is such as to allow of localized ductility the effect is to produce irregularly shaped faults or depressions on flat surfaces which show very clearly and which are very difficult to remove in ordinary grinding and polishing procedure.

EFFECTS OF VARIOUS ELEMENTS

In the following paragraphs the effects of certain added elements upon various properties of aluminum die-casting alloys are briefly stated. No pretense of completeness is intended—at best the information can be but general.

Effects of Zinc:

Zinc in aluminum alloys causes hot shortness. In pressure die casting it must be kept low in proportion to the nature of the casting being made. In same cases as little as 0.25 or 0.5 per cent will ruin the alloy; in other cases as high as 1.25 to 1.50 per cent is permissible. In so far as is known zinc in percentages up to 1.5 per cent is not harmful to the mechanical properties of the finished casting. There is some indication that this amount in copper-aluminum alloys is even beneficial in that it increases strength and machineability at room temperatures.

Effects of Nickel:

Nickel in aluminum alloys in proper percentages is thought by some to increase strength at elevated temperatures, to cut down run marks or flow lines or cold shuts in the finished casting, to cut down the avidity with which the molten metal attacks iron and steel machine and die parts, and to offset to some extent the effects of iron in the alloy. Nickel seems to have the property of increasing the strength and hardness of the finished casting in much the same way as does iron, but without the development of excessive brittleness which would accompany an equal percentage of iron. Nickel also seems to give improved polishing properties and greater light reflectivity to the finished castings. Nickel does not seem to have the deleterious effects of iron on high-silicon alloys. In short, nickel is often a solution of a problem in production, is usually an advantage and seldom the cause of trouble to the consumer. Nickel may have some effect on corrosion resistance. Nickel plate, for example, seems to promote the corrosion of aluminum, but it is a question whether nickel in the alloy itself has a similar effect. No authentic data are available, but the indications are that it is not nearly so active when alloyed with the aluminum and that in the alloy it is not so active in promoting corrosion as an equal percentage of copper.

Effects of Silicon:

Silicon when added to quite a number of aluminum alloys seems to decrease shrinkage and increase ductility at elevated temperatures. An addition of 1 or 2 per cent of silicon or a 1 or 2-per-cent increase in silicon content of the alloy very often prevents cracked castings in production. No authentic data are available as to the effect of the addition of 2 per cent of silicon to copper-aluminum or copper-nickel-aluminum alloys for pressure die castings. The author has found, however, that such additions have a tendency to harden the alloys somewhat and should be compensated for by a corresponding reduction in copper or copper and nickel content. When such compensation is made, it seems that there has resulted no material change in the mechanical properties of the finished castings.

Effects of Iron:

Iron plays a rôle in connection with hot shortness and strength and ductility at elevated temperatures. Its use, however, must always be limited by the effect it has on the properties of the finished casting. In many alloys, iron in proper quantities seems to decrease hot shortness and increase strength at elevated temperatures, and to cause the stretching to be more evenly distributed over the entire casting. Excessive iron, however, sometimes causes hot shortness and brittleness. As other alloying constituents such as copper, nickel, silicon and manganese increase, the maximum and minimum permissible iron content for efficient production of a complicated or difficult casting decreases.

The iron range does not seem to be definite for a given alloy over a large variety of different castings. In practice the producer does not make numerous chemical analyses of castings in order to locate this range. Analyses are costly and slow and require technical men to interpret them as well as to make them. Iron is usually controlled in the very practical way of proportioning the relative amounts of new alloy and remelt used in the casting machine. Sometimes all remelt and sometimes all new metal is found to give best results. This may be an unscientific method of controlling iron but a very practical and efficient one. Sometimes when large proportions of remelt are necessary in production the final casting is too hard or brittle. This can often be corrected by adding to the remelt some virgin aluminum, thus diluting it and reducing not only the iron content but also the content of the other hardening elements present. The type of alloy used then remains the same as the original alloy, but the iron content has been increased while the content of the other alloying constituents has been decreased. If the desired mechanical properties of the finished casting are maintained there should be no objection on the part of the consumer to such variations.

In a later section of the paper the iron content situation is discussed more particularly from the standpoint of the problems of production.

Effects of Magnesium:

Magnesium in amounts of 0.10 per cent seems to decrease fluidity, increase shrinkage, cause some additional hot shortness and decrease strength at elevated temperatures. It is, therefore, very important from the producer's standpoint that magnesium be carefully controlled. In some castings its presence must be avoided, in others 0.05 per cent is allowable and in others 0.10 per cent. Seldom, however, is it desirable. Magnesium also has a marked effect on the mechanical properties of the finished castings. Initial hardness is increased, some age hardening often results, ductility is greatly decreased. The author knows of a case where the presence of 0.10 per cent of magnesium in an alloy of the 4-per-cent nickel, 4-per-cent copper, 1.5-per-cent silicon type made the castings absolutely unfit for the use intended. In many cases 0.05 to 0.10 per cent of magnesium will make good surface finish impossible.

Effects of Tin:

Tin decreases the strength and ductility of aluminum alloys at elevated temperatures even more than at room temperatures; in other words it causes hot shortness. This can be partially overcome by the liberal addition of silicon to the alloy, the amount of silicon necessary being usually three times the amount of tin added; but even then the alloy causes trouble due to hot shortness. Tin improves the machineability of aluminum alloys, as discussed in more detail in a later section of the paper.

Summary of Effects of Different Elements:

In Table I the author has endeavored to summarize qualitatively the effect of each of the metals ordinarily found in aluminum alloys for die casting. This information is very general only and has been gleaned from observation of actual production. In combinations of two or more added elements the results are sometimes very different than are indicated in the table. The effects given in the table are those apparently produced by relatively small additions of the elements; larger additions often act entirely differently. In a few cases this is shown qualitatively in the table by (− to +), etc., meaning that as the quantity of added element is increased its effect changes.

TABLE I.—EFFECT OF ADDED ELEMENTS ON PROPERTIES OF ALUMINUM ALLOYS.

+ indicates an increase in the property with increase in percentage of the element. Thus, fluidity is increased as the percentage of copper is increased.

− indicates a decrease in the property with increase in percentage of the element. Thus, shrinkage is decreased as the percentage of copper is increased.

− to + indicates a change from decrease to increase in the property with increase in percentage of the element. Thus, as the content of iron is increased its effect on shrinkage changes from decrease of shrinkage to increase of shrinkage.

+ to − and 0 to − are to be read similarly.

Elements	Fluidity	Shrinkage	Iron Attack	Hot Shortness	Strength at Elevated Temperatures	Machine-ability	Polishing Properties	Plating Properties
Copper	+	−	−	−	+	+ to −	+	+
Silicon	+	−	+	−	+	0 to −	−	−
Iron	−	− to +	−	− to +	+	+ to −	+	+
Nickel	−	− to +	−	−	+	+ to −	+	+
Zinc	+	+	0	+	−	+	−	−
Manganese	−	−	−	−	+	+ to −	+	0
Magnesium	−	+	0	+	−	−	−	−
Tin	+	+	−	+	−	+	−	−

In this table are also shown columns for machineability, polishing properties and plating properties. After the castings are removed from the die they must go through cleaning and finishing operations. Sometimes this means only the removal of fins and gates by band sawing and filing, but more often some machining operations are performed; quite often some grinding and polishing, and occasionally plating, operations needed to be performed, as discussed below.

Effects of Added Elements on Mechanical Properties:

There is very little information on the effects of various added elements on the mechanical properties of the different aluminum

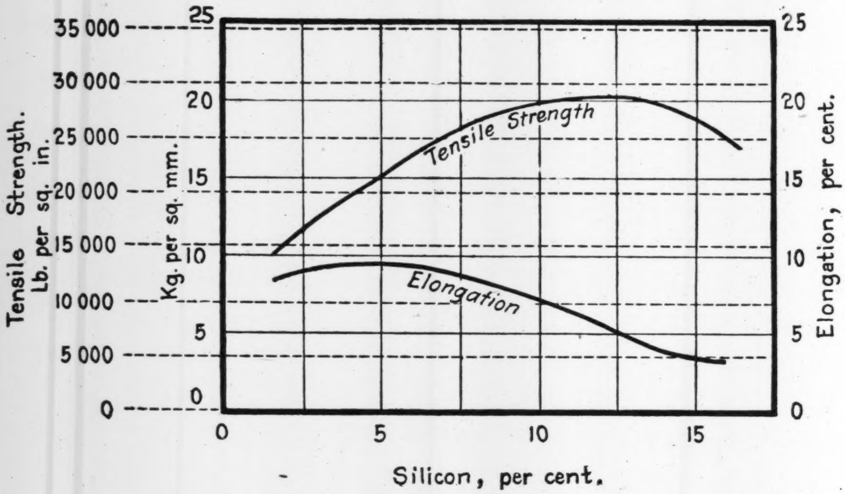

FIG. 1.—Effect of Silicon, on the Physical Properties of Silumin.

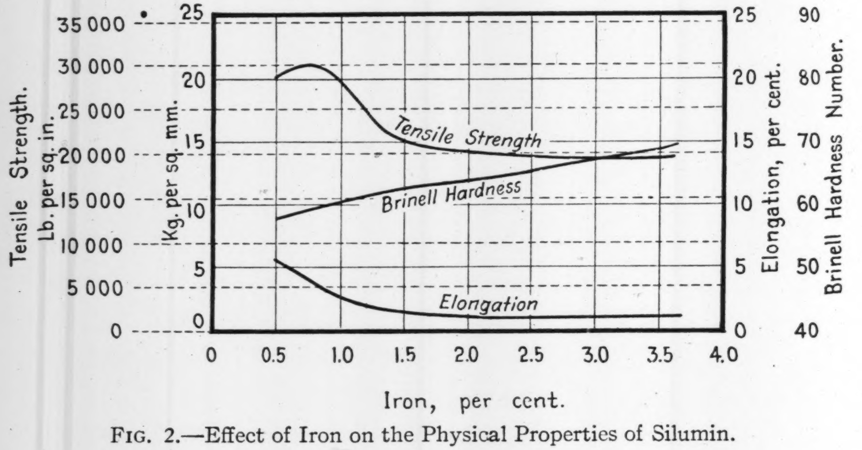

FIG. 2.—Effect of Iron on the Physical Properties of Silumin.

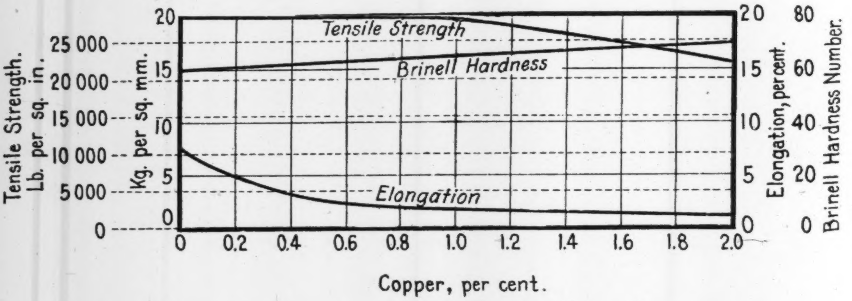

FIG. 3.—Effect of Copper on the Physical Properties of Silumin.

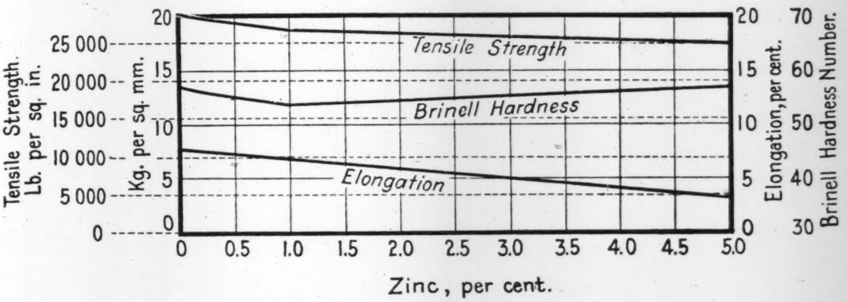

FIG. 4.—Effect of Zinc on the Physical Properties of Silumin.

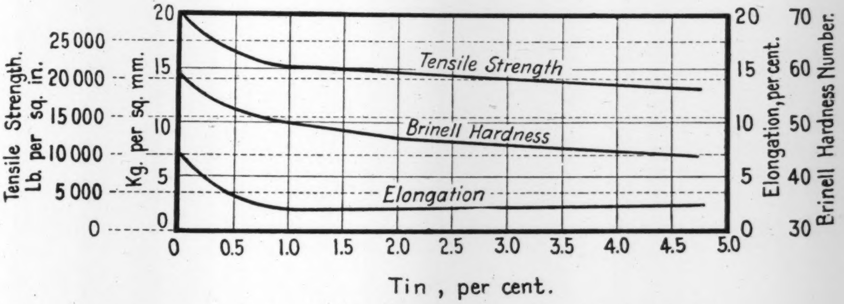

FIG. 5.—Effect of Tin on the Physical Properties of Silumin.

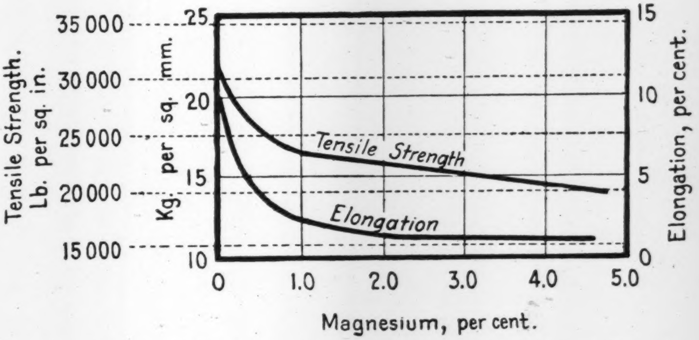

FIG. 6.—Effect of Magnesium on the Physical Properties of Silumin.

alloys, which information is greatly needed. The advantages of co-operative effort in obtaining such data are obvious and tests made in a number of laboratories on castings prepared by different producers in which the percentages of various elements are carefully controlled, would be of the greatest value. Some work of this nature has been done in the sand-casting field.

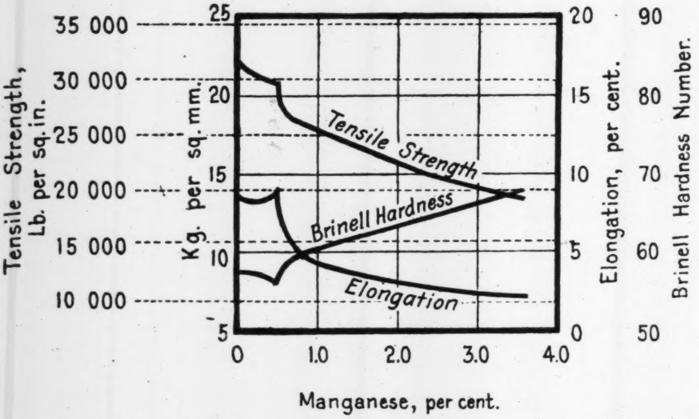

FIG. 7.—Effect of Manganese on the Physical Properties of Silumin.

NOTE.—Attention is called particularly to the reversal of slope at approximately 0.5 per cent manganese.

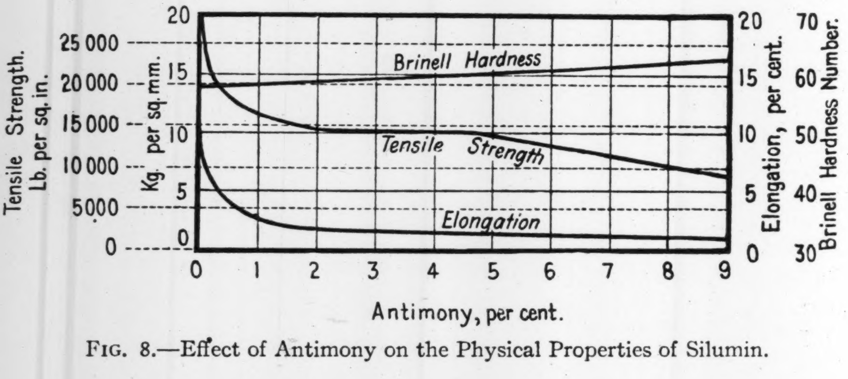

FIG. 8.—Effect of Antimony on the Physical Properties of Silumin.

Mr. F. J. Tobias[1] has supplied the curves shown in Figs. 1 to 8, taken from an article[2] by Joseph Dornauf of Frankfurt, Germany. These curves show the effect of silicon, iron, copper, zinc, tin, magnesium, manganese and antimony on the tensile strength, elongation, and in some cases hardness, of the alloy silumin, 13 per cent silicon, 87 per cent aluminum, as sand cast. Mr. Tobias states that he has personally checked these data in some cases against pressure die castings of a similar composition and has found a close relationship to exist.

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[1] F. J. Tobias, Research Dept., Newton Die Casting Co., New Haven, Conn.

[2] Joseph Dornauf, "Silicon and Aluminum Sand Castings," *Zeitschrift für Metallkunde*, August, 1928.

While data from these curves may not be directly applicable to pressure die castings, they do show quite conclusively the serious effects of iron, copper, tin, magnesium and antimony, and the minor effects of small amounts of zinc and manganese in line with the previous remarks in the present paper. Particular attention is called to the reversal of slope at around 0.5 per cent of manganese in the curve showing the effect of manganese on the 13 per cent of silicon aluminum alloy (Fig. 7). Data of similar character for aluminum alloys in the form of pressure die-cast bars would be of great value.

MACHINEABILITY AND HARD SPOTS

Machineability is often a source of difficulty with aluminum die-castings. They can be too soft and stringy and therefore difficult to machine. They may be too hard and therefore difficult to machine. They may be full of hard spots which quickly take the cutting edge off the tool. By proper grinding of cutting tools the trouble due to the first two sources can usually be overcome, but scarcely anything can be done in respect to castings so full of hard spots as to act like grinding wheels on the cutting edge of a tool.

There seems to be the general impression that high iron content is the basic cause for all machining troubles. Obviously this is not the case where the difficulty is due to the metal being too soft and stringy, as iron in considerable amounts increases the hardness of the alloys. This is not so obvious in the case of alloys which are difficult to machine due to high hardness. However, practically all alloy additions to aluminum except tin and zinc, and possibly silicon in some instances, increase hardness as it affects machineability. In other words, this difficulty in machining may be due to high alloy content other than iron. For example, 0.2 per cent of magnesium will often have as great an effect as 2 per cent or sometimes even 3 per cent of iron.

Often hard spots are accompanied by high iron, but is there any direct relation between high iron and hard spots? This depends upon the composition of the hard spots. A simple and quick test can, in many cases be made to determine if the inclusions are metallic iron, iron oxide, iron silicide, or some other iron compound. If the clean machined surface showing one of the hard spots is treated with a solution of potassium ferricyanide containing a little sodium chloride a blue scum will form in a minute or two over the hard spot if the spot is iron or an iron compound subject to attack by a sodium-chloride solution. In a vast majority of cases hard spots are not due to high iron but are simple inclusions of aluminum oxide, dross, or other foreign materials. High iron, however, often accompanies hard spots caused by overheating and long stewing of molten aluminum in an iron pot. By proper fluxing of the metal the hard spots or inclusions can usually be removed and the fluxed metal, although high in iron, will give no trouble due to hard spots. The author, for example, has seen 3 to 4-per-cent copper-aluminum alloy containing as much as 6 per cent of iron machined without trouble. Hard spots can be caused by overheating and long stewing of molten aluminum in graphite crucibles where no iron pick-up occurs. They may be carbides, silicides, clay, sand, brick or any of a number of things. They may be direct evidence of poor alloying practice in the addition of metallic silicon or hardeners to the aluminum.

Tin and zinc both increase machineability of aluminum alloys. Tin is much more effective than zinc, and acts apparently very similarly to lead in brass alloys. The addition of only 0.5 per cent of tin is often sufficient to give free machining properties. Sometimes more is needed, and the author has seen aluminum die castings made with tin as high as 4.0 per cent. The advantage gained by additions of tin beyond 2.0 per cent are very slight as regards machining, while the disadvantages as regards casting properties of the alloy become considerable.

POLISHING PROPERTIES AND PLATING

Polishing properties are important in many instances. Very soft metal is much more difficult to polish than hard metal. The luster and reflecting power of the polished surface vary with the composition of the metal. Silicon deadens the luster and decreases reflectivity, as also does tin. Copper, nickel and iron all improve polishing properties, brighten the luster and increase reflectivity. Porosity (blowholes and sponginess) and hard spots are ruinous to polishing properties. Magnesium seems to be harmful to polishing properties; manganese seems beneficial.

Nickel plating of certain aluminum-base die castings has been a quite regular practice for a number of years. Corrosion resistance and appearance have been the objects sought. More recently cadmium plating for corrosion resistance alone has become fairly common practice. At present several companies are attempting to chromium plate over copper and nickel with more or less indifferent results. Considerably more experience must be had before much can be said about the relative properties of different aluminum alloys as regards cadmium and chromium plating. In nickel plating it seems that copper, nickel and iron additions, in the order named, improve plating properties, while silicon, zinc, magnesium and tin are detrimental.

THE IRON CONTENT SITUATION

The problem of using up scrap, gates and runners is a vital one in the pressure die-casting industry. As explained before, every time the alloy goes through the process certain impurities are picked up, mainly iron to the extent of 0.5 per cent. Sometimes the weight of gate plus runners plus overflows and fins will be ten times the weight of the finished casting, while in some cases it may be as low as one-half or one-third. The average ratio of weight of finished castings to total weight of metal actually cast is of the order of 1 to 3. This means that the finished castings are made of metal which has been through the casting machine an average of three times. This means that three times 0.5 per cent or a total of 1.5 per cent of iron has been picked up. This added to the original 0.5 per cent of iron as an impurity in the metals used, plus the 0.3 per cent picked up in alloying in iron pot furnace practice, means a total average iron content in finished castings of 2.3 per cent. A high iron content is obviously inherent in the process.

By use of refractory-lined alloying furnaces, the iron pick-up due to alloying can be eliminated. By careful control of casting temperatures and gating and venting so as to permit of as low casting temperatures as possible, the iron pick-up in the process might be reduced to 0.4 per cent or even as low as 0.3 per cent per operation. By a redesign of casting machines and development of better die steels, gates and runners could be made smaller and shorter and the average ratio of weight of finished castings to weight of metal cast might be brought down to 1 to 2.

The result of all these combined would be an average iron content of finished castings of 1 to 1.15 per cent. But these improvements have not yet materialized. Refractory-lined furnaces for alloying are gradually being adopted but are by no means universally used in the industry. Not much is being done in respect to gating and venting as related to casting temperatures. Redesigning of machines and replacing of equipment is a very slow process. Under present conditions, therefore, we must expect an average iron content of 1.75 to 2.0 per cent as an optimum in finished aluminum pressure die castings.

But this iron content is too high for certain alloys, such as the high-silicon alloys and for certain special castings. Every die casting company has certain customers who specify low iron content in their castings, and who even demand that only new metal—no gates or scrap—be used on their work. These same customers usually refuse to consider paying an additional charge for this purity of metal. If say 20 per cent of production has iron below 1.5 per cent, say an average of 1.3 per cent, the result is that the remaining 80 per cent of production must average 2.5 per cent of iron.

The above is rather a startling picture of the iron content situation as faced by the aluminum pressure die-casting industry, but it is not at all hopeless. Certain additions to silicon-aluminum alloys, for example, have been developed recently which seem to counteract or neutralize the effects of iron on those alloys. They are being tried out, although they have not yet come into general use. A partial separation of iron from molten aluminum alloy takes place on slow cooling and freezing, a high iron-bearing sludge settles to the bottom and on careful remelting and pouring off, some 80 per cent of the alloy can be recovered containing only about 1.25 per cent of iron, the remaining 20 per cent being molten alloy still rather high in iron plus the sludge which may run from 10 per cent up to as high as 25 per cent of iron. This method of treatment requires special large furnaces, is costly and time consuming. The high-iron-bearing sludge has no ready market and represents a considerable direct loss in metals to the producer. In the present day of keen price competition, the adoption of such processes is naturally very slow.

The proper picture of the iron situation might be summed up as follows. The average iron content of aluminum alloy pressure die castings can be expected to be slightly above 2 per cent. If a maximum of 2 per cent permissible iron is set, a slight premium should be paid for metal purity. If the maximum is set at 1.5 per cent, a considerable premium should be paid for metal purity.

CONCLUSION

In this article the author has tried to outline the alloy problems of the aluminum pressure die casting industry. Specific data in support of many of the statements made are not available. Some of the impressions that have been gained may be in error due to some unaccounted-for variable having at times produced a condition which has been ascribed in error to some other variable. It is hoped that further work by others will lead to proving or disproving these impressions, for very few of them can be set forth as facts at the present time.

DISCUSSION

Mr. D. L. Colwell[1] (*presented in written form*).—The writer wishes to congratulate Mr. Tour on being the first to come out in print with "impressions," as he calls them, based on long experience in die-casting aluminum, and at the same time to express a belief that Mr. Tour's conclusions are substantially correct. The description of the die-casting process gives the reader an excellent idea of just what happens in the die, and is necessary for a proper understanding of the effects of various elements.

The following comments on each of these various elements may be of interest:

Zinc.—The effects of zinc are about as stated. Since most die casters cast both zinc-base and aluminum-base alloys, contamination of the aluminum by zinc is a circumstance which must be recognized. Because of its harmful effects on casting properties, the producers usually keep it at a figure below 0.75 or 1.0 per cent. It might be dangerous to the consumer in larger amounts by causing casting defects not discovered by inspection; and so in general is undesirable except in small quantities.

Nickel.—Mr. Tour considers nickel a desirable addition, except, perhaps, in promoting corrosion. It has been our experience that nickel is considerably less active in promoting corrosion than a similar amount of copper. It should be, therefore, a desirable addition in every respect. Its only disadvantage is its price.

Silicon.—Mr. Tour's remarks could well be extended to include higher percentages of silicon.

Iron.—The control of iron content in aluminum alloys by varying the proportions of new metal and remelted metal is certainly a scientific method. Any other method of reducing iron would be impractical, and any other method of adding iron would be poor economy. The relation of high iron to machineability is accurately discussed, and we believe the author's conclusions to be sound.

Mr. Tour's closing paragraphs on the iron content situation portray a somewhat gloomy picture for the die caster. It has been our experience that his figures are a little bit high, although his method of reasoning is sound, and that die castings can be produced year in and year out with the "optimum" percentage of 2.0 per cent iron as a maximum. If, however, a consumer should demand a 1.5 per cent maximum on iron content, he should certainly be willing to pay a premium.

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LIFE TESTS ON METALLIC RESISTOR MATERIALS FOR ELECTRICAL HEATING

By F. E. Bash[1] and J. W. Harsch[2]

Synopsis

In view of the growing importance of electrical heating and the use of metallic resistors, it was considered important to devise a durability test that would be a measure of the quality of the material used in the heating unit as regards length of useful service. It was felt that an accelerated test was the only kind that would be of much value.

There are two general types of service: (1) electric appliances, such as toasters and electric irons; and (2) electric furnaces. The appliance is run for short intervals at constant voltage and the furnace for long intervals at constant temperature.

Tests were made on the effects of all variables encountered in a life test, the most important of these being:

1. Temperature;
2. Heating and cooling cycles;
3. Mounting,
    (*a*) position,
    (*b*) form;
4. Dimensions of test specimen;
5. Atmosphere; and
6. Voltage variation.

All previous types of tests were considered.

It was found that an accelerated test of 100 hours average duration could be made by running the specimen under test at a comparatively high temperature at constant voltage and by heating and cooling it in cycles of 4 minutes—2 minutes on and 2 minutes off.

The life tests were established after many comparative tests had been made on the same wire in five different laboratories. After the variables had been brought under control, final tests were made in three different laboratories in which agreement among them was obtained within ±6 per cent for one class of alloy and ±2 per cent on another.

The authors are satisfied that an accelerated test on electrical resistor wire can be made which will be reproducible and believe it will give results which are a measure of the quality of the material tested.

The growth of the electrical industry and the development of electrical power has greatly increased the use of electrical heat and has

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emphasized the importance of high-grade resistor alloys. A great deal of attention has been given to the question of design and the application of these resistor materials, while comparatively little study has been made of the characteristics of the alloys themselves and methods of determining quality. In the application of these alloys it is important to know not only their physical and chemical properties but also how the materials will behave under service conditions, especially in cases where high temperatures are involved. The performance of these wires cannot be predicted unless they have been tested at the elevated temperatures for which service they are intended. It is, therefore, important to devise some accelerated test at elevated temperatures which will give this information. Life tests are now being made on many classes of material in varied lines of industry and are proving their value daily, the automotive industry being an outstanding example. Life tests of electrical heater wire should give equally satisfactory results.

There are three factors which govern the quality and satisfactory service of heating elements: (1) design; (2) conditions under which they are used; and (3) the materials from which they are made. The question of design of apparatus in industrial heating devices and appliances is one which concerns chiefly the company manufacturing them. It is outside the scope of this paper to consider features of design except in a general way.

The conditions of use are important. It is known, for example, that certain materials such as the salts of alkali metals have an injurious effect on nickel-chromium wires when hot. The salts apparently flux the nickel and chromium oxides and form low-melting compounds which fuse and uncover the surface of the metal beneath, allowing it to be attacked rapidly and in a short while causing a hot spot and burnouts. Other substances corrode the heating elements by forming compounds with the constituents of the alloy. It should be borne in mind in designing apparatus and appliances that all corrosive materials should be kept out of contact with the heating elements.

Materials which are used for the heating elements may be metallic or non-metallic, although the great majority are metallic. The principal characteristic of the metallic materials must be that they resist oxidation at the temperature to which they are subjected in service. These materials may be grouped into three classes as follows:

1. Metals or alloys which are not subject to oxidation, as for example, platinum. The use of such materials is practically prohibited due to their high cost.

2. Alloys which are readily oxidizable but which form a protec-

tive coating of oxides of the constituents in the alloy on the surface. It has been found that the clean surface of properly melted and processed nickel-chromium alloys and nickel-chromium-iron alloys will oxidize very readily when subjected to heat, and that this oxide is very adherent and is comparatively impervious to the penetration of oxygen. This, therefore, protects material under the outside skin from further oxidation.

3. Metals and alloys, as for example, molybdenum and tungsten, having a high melting point and which are readily oxidizable, the oxides of which do not form a protective coating but which can be used in strong reducing atmosphere.

Of the three types of materials listed above, those of the second group are of the greatest commercial importance and will constitute the subject matter of this paper, which covers work that has been done by the Society's Committee B-4 on High-Temperature and Electrical-Resistance Alloys, with special reference to durability or life of the nickel-chromium resistor materials at high temperatures.

It is undoubtedly of great importance to the consumer to have as high grade a material as possible for the construction of electrical appliances and furnaces. The difficulty has been in the past that there have been very few tests which could be applied to determine the desirability of any particular grade or make of alloy without testing the completed unit as installed in the appliance, oven or furnace. Such tests are long and expensive and are greatly influenced by design and conditions of use.

It is very desirable from the standpoint of producer and consumer that there be some scale by which to gage quality and it is particularly desirable also that any test which is applied be capable of standardization and be fairly easy of application. Such a test is desirable from the standpoint of the producer, as he will then be able to improve his product and determine the effect of the variables in his manufacturing processes and variations in composition. He will also be able to compare new alloys with those already being produced. Furthermore, it is desirable from the standpoint of the consumer, as he will be able to have a check on the quality of material furnished, on comparative values of competing materials, and new materials as they are introduced.

Committee B-4 has been working for over three years on the subject of standardized tests for quality and durability. It has accumulated a quantity of data in regard to life tests, and sufficient information has now been gained to give a clear idea of desirable and undesirable features and the effects of different variables upon the life of electrical heating materials.

VARIABLES AFFECTING A LIFE TEST

Types of Service:

There are two general types of service in which a heating unit may be used; the electrical appliance which is used for short periods of time and in which the heating and cooling is very rapid, and the electric furnace where the unit is under heat for long periods of time with slow rates of heating and cooling.

Comparing these two types of service at the same temperature the appliance type is the more severe due to its rapid heating and cooling. Starting with wire or strip that is clean and free from oxide, when the current is turned on an oxide coating will be formed on the wire immediately upon first reaching a red heat. This gradually becomes thicker as time goes on. As the oxide becomes thicker, due to continued use, the repeated heating and cooling causes a variable amount of cracking and scaling of the oxide, depending on the quality of the alloy. Any metallic surface exposed by this scaling immediately forms a fresh coat of oxide. The adherence and imperviousness of this oxide coating determines the effective life of the heating element. Good adherence prevents any large amount of scaling and consequent exposure of fresh metallic surfaces, while imperviousness of the oxide coating prevents penetration of oxygen to the metal. A permanent impervious and tight coat of oxide effectively blocks further oxidation with its consequent reduction in cross-section and increase in resistance.

Temperature:

Temperature is one of the most important variables in a life test. The life of a wire varies inversely with an exponential function of the temperature. This is readily understandable when it is considered that the life of the wire when operated at a temperature close to the melting point will be very short while at lower temperatures it will be practically indefinite.

We have found that it is a comparatively simple matter to determine the temperature of a wire having a diameter of 0.020 in. or greater by means of an optical pyrometer of the disappearing filament type with a magnifying objective lens. The desired magnification may be readily obtained by substituting for the standard objective lens another one having approximately half its focal length.

Cycles:

From the above discussion on types of service it will be apparent that cyclic testing, which consists of alternate heating and cooling of the material, offers a means of control of the duration of a life test. It has been found, for instance, that allowing the wire to cool once in 24 hours shortened the life to 50 per cent of a continuous test run on the same wire. Shorter cycles still further reduce the length of life of a wire.

Dimensions:

The size and shape of the specimen under test has a bearing on the number of hours of life. This has been established by a few preliminary tests, but has not been worked out in detail. It is expected that data on this feature will be gathered and reported at some future time. Tests made by the committee have all been on round wires having a diameter of 0.025 or 0.0285 in.

Mounting:

Many types of mountings have been used in the experiments. The wire has been tested straight, in the form of a helix, mounted in air and upon refractory supports. The straight wire has been mounted horizontally, in the shape of a catenary, and vertically. In the vertical position it has been rigidly connected at both ends and also suspended free in air with a small weight attached at the lower end. This latter method was finally adopted.

The disadvantage found in testing wire in helical form was that it is very difficult to so space the turns that every sample will be exactly like previous ones or like those mounted in other laboratories. Tests were made on straight wires in a horizontal position and it was found that there was a large temperature variation between the center and the sides, and that the wire usually failed close to one of the binding posts due to flexure at that point as the sample changed in length with heating and cooling. The catenary mounting was also tried and was abandoned for similar reasons.

It was found in the case of the vertical wire when it was rigidly connected at both ends that there was a certain amount of bending action on the wire due to expansion, so that it was found better to suspend the wire with the lower end free in order to get the effect of oxidation without uneven mechanical stress. The temperature variation along the length of a vertical sample has been found to be smaller than in the case of the horizontally mounted wire, the explanation apparently being that the convection currents are greater near the top than at the bottom and for that reason the heat is carried away at a more or less uniform rate, so that the top is not appreciably hotter than the bottom as might be expected.

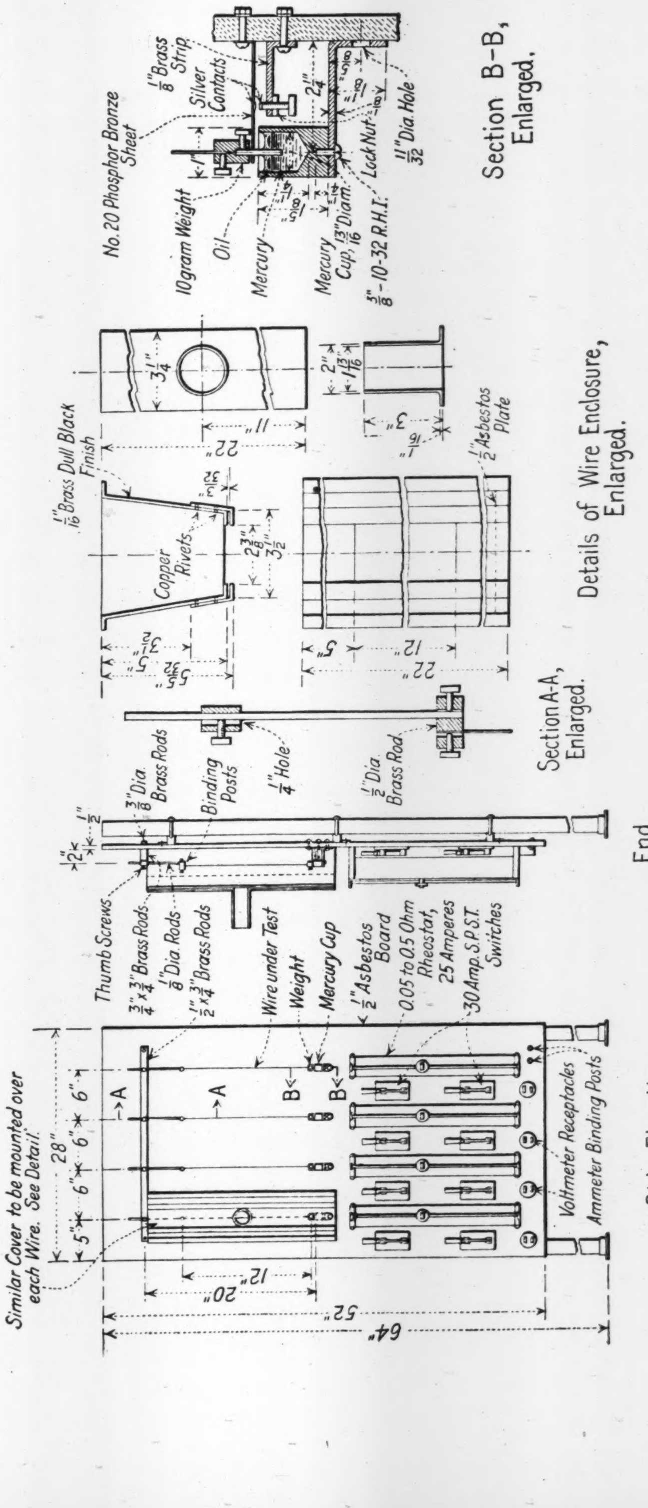
Similar Cover to be mounted over each Wire. See Detail.
Thumb Screws
3/4" x 3/4" Brass Rods
1/8" Dia. Rods
1/2" x 3/4" Brass Rods
Wire under Test
Weight
Mercury Cup
1/2" Asbestos Board
0.05 to 0.5 Ohm Rheostat, 25 Amperes
30 Amp. S.P.S.T. Switches
Voltmeter Receptacles
Ammeter Binding Posts
28"
6"
6"
6"
5"
12"
20"
52"
64"
A
A
B
B
Side Elevation.
3/8" Dia. Brass Rods
Binding Posts
2"
1/2"
End Elevation.
1/4" Hole
1/2" Dia. Brass Rod
Section A-A, Enlarged.
1/16" Brass Dull Black Finish
Copper Rivets
3/32
2 3/8"
3 1/2"
3 1/2"
5"
5 5/32
5"
12"
22"
22"
11"
3 1/4"
2"
1 13/16"
3"
1/16"
1/2" Asbestos Plate
Details of Wire Enclosure, Enlarged.
No. 20 Phosphor Bronze Sheet
10 gram Weight
Oil
Mercury
Mercury Cup, 13/16" Diam.
5/8"-10-32 R.H.I.
1/8" Brass Strip
Silver Contacts
1"
2 1/4"
Lock Nut
11/32" Dia. Hole
Section B-B, Enlarged.

FIG. 1.—Test Board for Accelerated Life Test.

Atmosphere:

The condition of the atmosphere surrounding the sample has been found to be very important, particularly in regard to currents of air. It was found at first that it was very difficult to obtain agreement in tests between different laboratories, due to the fact that there was a difference in the position of the test boards in respect to the walls, doors and windows. This caused a greater amount of draught in some cases than in others, with the resultant effect upon the rate of cooling and the variations in the operating temperatures of the test specimens. Considering the fact that temperature and rate of cooling are the two most important factors of the test with respect to their effect upon the length of life of the wire, it can readily be seen that the effect of draughts must be minimized and if possible entirely eliminated. It has been found that the effect of draughts upon the life of a sample of wire may amount to a 50-per-cent variation.

It is also necessary that the atmosphere be free from fumes which might attack the wire. For this reason it is not desirable to set up a test board in a chemical laboratory.

On account of the variations due to draughts, it was found necessary to enclose the test specimens with metal covers which are described in the proposed Tentative Accelerated Life Test for Metallic Materials for Electrical Heating,[1] being recommended by Committee B-4 (Fig. 1), in order to get uniformity of results between laboratories.

Mechanical Condition of Sample:

It has been found that any kinks or surface imperfections in the test wire have a tendency to produce localized over-heating and consequent failure at that point.

Voltage Variations:

Voltage variations produce corresponding variations in wattage and the resulting temperature. From the above discussion of effect of temperature it will be evident that a close control of voltage is, therefore, necessary. It is, of course, possible to make life tests where a number of specimens are run in parallel and get comparative results between a number of samples. It would be impossible, however, under such conditions to get consistent and comparable results over any period of time or between different laboratories.

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[1] See *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part I, p. 613 (1929).

TESTS PREVIOUSLY USED

There are some published data on life tests and specifications for tests on high temperature resistor materials, which are briefly summarized below:

The British Standard Specifications for Metallic Resistance Materials for Electrical Purposes, No. 115-1924, published by the British Engineering Standards Association, specify a test for materials to be used at 700 and 1000° C. as follows:

> *Class D.*—The material shall withstand a temperature of 700° C. maintained continuously in a furnace in which the free access of air to the sample is not prevented, for four weeks. After the conclusion of the test the resistance shall not have changed from its original value by more than plus or minus 2.5 per cent.
>
> *Class E.*—The material shall withstand a temperature of 1000° C. maintained continuously in a furnace in which the free access of air to the sample is not prevented, for four weeks. After the conclusion of the test the resistance shall not have changed from its original value by more than plus or minus 2.5 per cent.

The chief criticism of this test, based on our experience, is that the heat is supplied from an external source which does not develop the inherent defects in the wire. The cause of failure of a resistor wire is practically always a hot spot. As previously stated, this hot spot develops at first due to either non-uniform oxidation along the length of the unit or due to a surface imperfection or non-metallic inclusion. As the hot spot develops the resistance at that point increases, which causes a greater evolution of heat, and as the temperature rises, greater oxidation as compared with the rest of the unit takes place, so that eventually the temperature mounts to the melting point and the unit fails. The test as specified only shows the effect of continuously applied heat as regards surface oxidation.

Yuhatiro Utida and Makoto Saito[1] described an oxidation test of nickel-chromium wires heated at different temperatures in an electric furnace and the increase in weight recorded with time. As an indication of quality or service of wire in an appliance this test is open to the same criticism as the British Engineering Standards Association test described above.

In addition to the published tests described above, there are innumerable so-called "quality tests" which have been made by manufacturers, a few of which are described below.

A chemical analysis requirement is common and is necessary to outline the class of alloy which it is desired to purchase. Beyond that it is apparently valueless as an indication of life in service. We have encountered numbers of instances where two or more alloys would have the same analysis within reasonable limits and we find the life to vary greatly. The chemical constituents which reduce the life of a heating element are either present in such small quantities that they are not detected or no analysis is made for them in a standard analysis. We have particularly in mind oxygen or dissolved oxides.

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[1] Yuhatiro Utida and Makoto Saito, "The Oxidation of Metals and Alloys at High Temperature," *Science Reports*, Tohoku Imperial University, Sendai, Japan.

A second test which is commonly made is to connect in series two or more wires of approximately the same size in an electrical circuit and to vary the current through them until one fails. This test may require from a few seconds to a few minutes. This is obviously a very unfair test as a slight difference in size or temperature coefficient of the wires may cause the burnout of the superior wire due to slight differences in temperature near the melting point.

A third test is to take two or more competing wires, form them into helices, heat them for a specified time at the same temperature, and then in each case to stretch the helices and catch and weigh the scale from each sample. This test is only a measure of the tightness of the scale and is not a true measure of durability or life since it does not take into consideration a number of important factors mentioned below.

A fourth test is to take two wires with approximately the same dimensions and cold resistance and mount them in parallel in two different appliances of the same type. A constant voltage is impressed and maintained over a long period of time and the increase in resistance is noted at regular intervals. While this type of test has certain advantages, the temperature of the units, which is a very important variable, is not controlled and there are numbers of variables which might affect one and not the other due to the design of the appliance.

One consumer's requirements for two grades of nickel-chromium and nickel-chromium-iron alloys which are to be used at temperatures of 800 and 1000° C. respectively, are as follows:

> The wire shall be heated to the temperatures called for for 20 minutes and cooled for 20 minutes through 1000 cycles, and after that treatment that they shall not show over 10 per cent reduction in tensile strength, and not over 60 per cent reduction in either its percentage elongation or its flexibility.

It would be much more desirable if this test were run to burnout with a record of the resistance changes throughout the test.

TYPES OF TEST

Any life test to be of value must be a measure of the actual or service conditions to which the unit is to be subjected. There are two types of tests which it would be desirable to make. One is a comparatively long test for investigative work. This test should bring out all the characteristics of any material and would be desirable from the standpoint of testing of a new alloy. The other test should be a short test which would be desirable for an acceptance test. This test could be applied to materials of the same class as supplied by any one or a number of manufacturers.

With these conditions in mind various types of tests were considered, as enumerated below.

Constant Temperature Test.—A test of this sort is run by adjusting the sample to a definite temperature at regular intervals and allowing it to run until failure. The resistance increase and other variables should be noted during the course of the test. The starting temperature would depend upon the size and type of alloy. This type of test is desirable for material which is to be used in electric furnaces which are maintained at more or less constant temperatures.

Constant Voltage Test.—This type of test is started at a definite temperature for a definite size and class of alloy and is then run under a regulated voltage to failure. The test is comparable to conditions in an electric heating appliance on line voltage. The temperature of the unit in this case decreases with time and the resistance increases.

A constant voltage test is the simplest test to make and is usually found to be as good a guide to quality as the constant temperature test.

Constant Wattage Test.—In this type of test the wattage or power is maintained constant. The chief difficulty with this test is that slight variations in temperature coefficient, and particularly in the type of surface of the wire, will have a considerable effect upon the temperature and will therefore not show true relative quality as between different materials.

Intermittent and Continuous Tests.—Due to the nature of the action of the protective oxide coating on nickel-chromium wires previously mentioned, the life of a wire continuously under heat will be several times as long as that of a wire which is heated for a short period of time and then allowed to cool.

The main consideration is that an intermittent test should be a guide to quality and should give results which are comparable to that of material in service. It is also desirable that the test should not be too long if it is to serve as an acceptance test. One hundred hours elapsed time is about the limit for the length of a short-time test.

The continuous test brings out qualities of the materials tested that will not be noted in an intermittent test. For example, it will show the rate of penetration of oxygen through the protective oxide coating, as compared with this effect combined with scaling due to rapid heating and cooling.

LIFE TEST REQUIREMENTS

After due consideration of all the above factors and after a large number of tests had been made covering the different variables, an accelerated life test procedure has been prepared which is being presented to the Society by Committee B-4[1] for approval as tentative.

It is believed that the tests and equipment outlined in the procedure represent the minimum outlay for apparatus necessary to obtain reliable results. For example, it is possible to make tests without voltage control and, under proper room conditions, without enclosing the sample. Experience has shown, however, that tests made under such conditions are not reliable. The test as outlined is adaptable for commercial testing laboratories which can make tests for the small consumer. Arrangements are now practically completed by one testing laboratory to make such tests.

Discussing the steps in order, the dimension of the test shield and general arrangements are given in Fig. 1, which is reproduced from the tentative test.

It is important that some type of current interrupter for the intermittent test be used which will work continuously with very little attention. A mercury switch in the primary of the transformer has proved to be satisfactory.

It is possible with the set-up and apparatus designated in Fig. 1 to make life tests on wire from No. 20 to No. 22 American wire gage. In case it is desired to make tests on other alloys having materially different specific resistance this can be taken care of by varying the length of the sample under test. For example, the wire having 25 per cent less specific resistance would have to be approximately 25 per cent longer or about 15 in. in order to carry the same current as is now required by the nickel-chromium alloys.

The accuracy of the temperature-measuring equipment is one of the most essential features of the test equipment. The instrument should be carefully checked at regular intervals to insure against errors. One method of checking the accuracy of the temperature measurement is the starting current required by any wire used as a standard of comparison. A series of tests will show that this wire usually requires a starting current which will be found to be quite constant. Any appreciable variations in this starting current would indicate the possibility that the optical pyrometer was out of calibration. At the start of the test the wire is usually unoxidized and bright in color and while it will take only a few minutes to acquire an oxide sufficient to make the emissivity for the temperature determination practically nil, it will require a number of hours for the surface to become stabilized so that the current temperature characteristics are more or less constant.

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[1] Tentative Accelerated Life Test for Metallic Materials for Electrical Heating, see *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part I, p. 613 (1929).

A number of methods of measuring the temperature other than by the use of the optical pyrometer were tried but none of them were found accurate enough to use even as an alternative method.

It has been found to be extremely helpful to select a specimen or coil of wire for any particular grade or class of alloy and reserve this as a standard of comparison. A series of tests can be run on this wire so that its life test characteristics will be established and it can then be run in parallel with new pieces of wire as a check on the general conditions and technique of operation.

In order to check the life test method during development, tests were made in five different laboratories on specimens cut from the same coil of Class *A* alloy (80 per cent nickel, 20 per cent chromium) wire and the results compared. After the main variables had been standardized and the technique established, it was found necessary due to the atmospheric conditions in the different laboratories to experiment with covers or shields for the specimens under test. A series of tests were made in one laboratory trying different types of covers and after a satisfactory one had been decided upon tests were run in four different laboratories to determine the comparative results between tests run enclosed and unenclosed. All of this work was done as stated above on the Class *A* alloy. A new series of check tests was then run in three different laboratories, the results of which are tabulated below. These tests are only a few of the many that have been run and are being run daily:

RESULTS OF TESTS ON ALLOY A WIRE (80 PER CENT NICKEL, 20 PER CENT CHROMIUM).

	NUMBER OF TESTS	LIFE, HOURS	AVERAGE DEVIATION, PER CENT
Laboratory *A*	4	115.5	±7.0
Laboratory *B*	4	106.4	±3.8
Laboratory *C*	8	118.7	±8.0
Average		114.8	

Another series of tests was then run on the Class *B* alloy (60 per cent nickel, 15 per cent chromium, 25 per cent iron) in the same three laboratories, with the following results:

	NUMBER OF TESTS	LIFE, HOURS	AVERAGE DEVIATION, PER CENT
Laboratory *A*........................	4	112.0	±1.8
Laboratory *B*........................	4	110.0	±7.8
Laboratory *C*........................	4	114.0	±1.3
Average..................................		112.3	

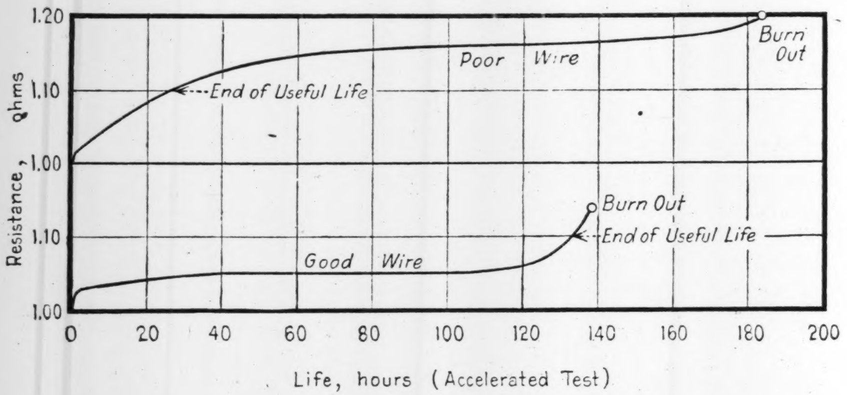

FIG. 2.—Results of Accelerated Test on 80 Per Cent Nickel - 20 Per Cent Chromium Alloy, Showing Relation of Life to Resistance.

These results show very good agreement and indicate that the method is satisfactory as regards ability to reproduce results. The test gives a quite reliable means of determining the relative quality of metallic electrical resistor materials. This will enable the design engineer to differentiate between the quality of material and other factors involved in design of electrical heating appliances or furnaces.

From a study of the results of tests made it is evident that the variation in the resistance of the material during test must be given consideration, since a wire which very quickly develops a high resistance might have an abnormally long life due to the drop in temperature resulting from the resistance increase. Thus a material of poor quality might show a long life if a high resistance develops quickly. Therefore, the nature of the variation of resistance with time must be known and is shown by curves giving the relation between time and resistance. This is brought out in Fig. 2. For the normal appliance or furnace a 10-per-cent drop in wattage or increase in resistance represents the end of the useful life of the element. For this reason the test procedure provides for determining the time to 10-per-cent increase in resistance as well as the time to burnout.

It has been found as a result of hundreds of life tests made over a period of 5 years that the material which gives the best results in service will have a resistance increase more or less rapid in the early part the test while it is becoming stabilized and will then very slowly increase in resistance with time.

Other factors aside from life and resistance increase which enter into consideration of quality are freedom from brittleness and absence of growth.

CONCLUSIONS

The authors feel that the use of the life test as outlined will be found to be beneficial to all concerned. The producer of the alloy will be able to make better material due to his having a measure of quality, and the manufacturer of heating appliances and the ultimate consumer will benefit thereby.

Acknowledgments.—The authors wish to acknowledge the assistance given by all the members of Committee B-4 and their respective companies and their cooperation in working out the methods of test here outlined. They also wish to acknowledge the preliminary work done by Mr. D. E. Foster and particularly wish to acknowledge the encouragement and cooperation of Mr. Dean Harvey who, as chairman of the Committee, has helped materially in the progress of the work.

DISCUSSION

MR. W. A. GATWARD[1] (*presented in written form*).—There seems to be no question but that the length of time required for a sample to undergo an increase of resistance of 10 per cent is a suitable measure of its usefulness as a material for heating elements. For any class of alloy there seems to be a more or less definite relation between the "useful life" and "total life." For the standard sample of alloy containing 80 per cent of nickel and 20 per cent of chromium, the useful life is approximately 55 hours while the total life as given by the authors is 115 hours. If a suitable test can be conducted in 55 hours, it might well be given a more prominent place. The total life, of course, should be investigated especially if an unknown alloy is involved. For known alloys, the useful life can be used as a measure of quality.

As shown in the accompanying table, the useful life data are more uniform than the total life data. The useful life is determined when hot spots are just starting to develop. Hot spots are rather "hit and miss." Some samples develop only one, while others from the same piece or wire will develop several hot spots. If only one hot spot develops, the resistance will not be greatly affected, but if several develop, then the resistance will increase more and will therefore reduce the current and the temperature.

The current and voltage across the sample at the "10-minute" adjustment will supply the hot resistance at the beginning of the test. In case the nature of the alloy is not known, readings should be taken frequently enough to supply data for a hot resistance *versus* time curve until the resistance has increased 10 per cent.

The accompanying data for Class A material covers 7 tests carried out according to the method proposed except that the samples were mounted on 3-in. centers and were exposed. Three samples were tested at a time, or 21 in all. The tests covered all seasons of the year, and no particular effort was made to standardize room conditions except that a barrier 6 ft. high was placed across one end of the test board and another across the front of the test board. The room was 20 by 40 ft. with windows across one end. These windows were open or shut, depending on the season of the year.

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[1] Chief Engineer, Hoskins Manufacturing Co., Detroit, Mich.

The mean "useful life" and the mean "total life" are given and the departure of each individual from the mean is stated. The numerical mean of the departures expresses the uniformity of the two values. The life values are correct to ± 1 hour.

The results show an expected error of 4.5 per cent for the useful life and 7.9 per cent for the total life. The fact that the samples were exposed does not affect the relative uniformity of the two classes of results.

TEST	USEFUL LIFE, HOURS	PERCENTAGE DEVIATION FROM MEAN	TOTAL LIFE, HOURS	PERCENTAGE DEVIATION FROM MEAN
No. 77	60.0	7.0	107.2	−4.1
	60.0	7.0	114.0	2.0
	57.0	1.6	114.5	2.5
No. 75	53.0	−5.6	122.0	8.4
	53.0	−5.6	100.7	−9.9
	56.0	−1.8	106.2	−5.0
No. 74	57.0	1.6	91.5	−18.2
	55.5	−1.1	98.0	−12.4
	54.5	−2.9	106.5	−4.7
No. 73	54.0	−3.7	Discontinued	
	49.0	−12.6	Discontinued	
	57.0	1.6	Discontinued	
No. 72	61.0	8.7	109.0	−2.5
	58.0	3.4	118.5	6.0
	62.0	10.5	118.5	6.0
No. 67	57.5	2.5	130.0	16.3
	52.5	−6.4	110.0	−1.6
	52.0	−7.3	124.0	10.9
No. 66	57.0	1.6	122.5	9.6
	56.0	−0.2	123.0	10.0
	57.0	1.6	97.0	−13.2
Means	56.1	4.5	111.8	7.9

The effect of temperature is the greatest source of variation. An error of 1 per cent in the original adjustment will cause a variation of about 10 per cent in total life and probably also on useful life. An accuracy of better than 1 per cent must be maintained if any closer agreement than 10 per cent can be expected.

The open test has several advantages. The space required is less. The samples are open for inspection and are accessible for measurement of diameter and length. The temperature of the samples can be compared easily and errors in readings of the pyrometer can be eliminated by visual inspection and any sample which appears too hot or too cold can be checked. The behavior of the samples can be observed during the test.

MR. DEAN HARVEY.[1]—Accelerated life tests for determining the durability of electric heating wires, of course, have been made for quite a number of years but until recently it has not been practicable to obtain a good check on the results, even though the tests were made in the same laboratory, to say nothing of comparing the tests made in different laboratories.

In developing the test method proposed, the members of Committee B-4 on High-Temperature and Electrical-Resistance Alloys were surprised to find what a large change in the results was caused by a comparatively small variation in temperature or by different current cycles on and off. The life of the specimen also was much shortened by a sharp bend in the wire. Close control of these various factors, however, has allowed a good check to be obtained even between different laboratories.

With the aid of this method as a means of judging the quality of their product, the producers of electric heating wires already have made considerable improvement in their products. We believe that considerably more progress will be made by the joint action of the producers and consumers as the test method is put into commercial use in the purchase of this class of materials.

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[1] Materials Engineer, Material and Process Engineering Department, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

THE DETERMINATION AND SIGNIFICANCE OF THE PROPORTIONAL LIMIT IN THE TESTING OF METALS

By R. L. Templin[1]

Synopsis

In view of the increasing tendency on the part of the users of metals to include values for proportional limit in specifications pertaining to such metals, it seems worth while to review both the significance of the proportional limit in design and the various factors involved in a satisfactory practical determination of this mechanical property.

The paper discusses in some detail the factors affecting the proportional limit and attempts to give some idea of the quantitative effects of the various factors on the proportional limit. Numerous stress-strain diagrams for a series of tension test specimens ½ in. in diameter, all prepared from the same material but tested under different conditions, are given to show the variations which may occur in the value for proportional limit due to internal strain in the metal, eccentric loading of the specimen, over-straining, the use of different though common types of extensometers and the use of different scales in plotting of the results.

A comparison is made of the proportional limit value and the so-called yield point value (strain corresponding to a departure of 0.2 per cent from the initial modulus line produced, under load). The available data indicate that the so-called yield point is in all probability of considerably more significance than the proportional limit due to the smaller errors involved in its determination. As a result of the discussion it would appear that the so-called yield point is appreciably easier to determine on a routine commercial testing basis than the proportional limit and probably is adequate for the needs of the designer and for product specification purposes.

A rigorous application of the mathematical theory of elasticity to the design of engineering structures made from metals usually involves among other assumptions that the metals deform under load according to Hookes' law up to a certain limiting stress value commonly called the elastic limit. In general, the elastic limit has been found by experiment to agree with the stress at which the stress ceases to be proportional to the strain, that is, the proportional limit. With the increasing tendency towards a more and more efficient use of metals in engineering structures, we are demanding more help from the higher mathematics and a more accurate knowledge of the mechanical properties of the metals used. In view of this situation it is not surprising to find a number of the users of metals wanting to know rather definitely the proportional limits for the various kinds, grades and tempers of the different metals.

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[1] Chief Engineer of Tests, Aluminum Co. of America, New Kensington, Pa.

Theoretically, the proportional limit of a metal may be considered important as representing a safe limit for the application of the usual mathematics and mechanics to the design of a structure made from the metal. Unfortunately, however, the practical determination of the proportional limit of a metal involves many factors, mostly difficulties, which apparently are not appreciated by many of the designers, makers and users of metallic engineering structures. The careful testing of a few specimens properly selected so as to be representative of a given metal, in order to define the proportional limit for that metal, apparently does not involve serious difficulties. The inclusion of a value for proportional limit in specifications for metal products, however, making it mandatory to determine this mechanical property on a routine basis for hundreds or thousands of specimens, not only causes a hardship which seems quite unnecessary but also leads us to inquire further into the usefulness of such results after they are available.

Some work has been done by this Society in attempting to standardize both the definition of and the method for determining the proportional limit of metals as evidenced by the Tentative Methods of Tension Testing of Metallic Materials (E 8 – 27 T).[1] In this tentative standard emphasis has been placed upon the "delicacy of methods and instruments used," that is, the sensitivity of the extensometer, the accuracy and sensitivity of the testing machine and the accuracy of the plotting of the test data. Other comments on the factors affecting the value of the proportional limit have been made recently before this Society, among which are notably those by Withey,[2] McVetty,[3] and Lessells[4] and those in the discussions by Thum, Menefee, Thackray and Tuckerman of Withey's paper, and Capp, Moore and Rawdon of Lessells' paper. In these papers and discussions mention has been made of many of the factors affecting the proportional limit but in view of the problem indicated previously it seems worth while to consider all of the factors affecting the value and attempt to obtain a better idea of the relative magnitude of the effect of each factor on the value of the proportional limit. We may then properly consider both the advisability of including this mechanical property in specifications for a metal and the significance of the values for proportional limit obtained under the imposed conditions.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 1067 (1927); also 1928 Book of A.S.T.M. Tentative Standards, p. 808.

[2] M. O. Withey, "Test of Specimens Cut from Different Portions of Structural Steel Shapes," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 41 (1928).

[3] P. G. McVetty, Discussion of a paper by M. F. Sayre on "A New Type of Mirror Extensometer," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 665 (1926).

[4] J. M. Lessells, "Concerning the Yield Point in Tension," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 387 (1928).

FACTORS AFFECTING THE PROPORTIONAL LIMIT

The factors affecting the proportional limit value may be classified under five major headings, as follows: (1) the material tested, (2) the test specimen used, (3) the testing apparatus used, (4) the testing procedure followed, and (5) the interpretation of the results obtained.

In general, metals are neither homogeneous nor isotropic but are essentially crystallin in structure and frequently possess a definite grain due to their method of fabrication. Other variations in their structure may be due to voids, seams, inclusions, grain size, segregation of constituents and uneven temper due in turn to variations in their method of manufacture. Still other variables might be cited for any given metal, but the point to be recognized is that any one of these many variables may and often does cause local yielding or even local failure in a test specimen cut from the metal, with the result that a change in the rate of deformation under load, of the specimen as a whole, occurs earlier than if the particular cause had been absent. That is, these variable factors affect the proportional limit of the material. With this fact in mind, the obvious intention in certain instances is to use the proportional limit as a criterion for determining the presence of such factors and their effects on the metal, without due consideration being given to the other factors involved in the determination of the proportional limit.

The test specimens used for the determination of the proportional limit will in most cases be the same as are used for determining the tensile or compressive strengths of a metal. Variations in dimensions, straightness and internal strain, resulting from the procedure used in preparing the specimens, which cause little if any differences in the ultimate strength or yield point values, frequently cause appreciable differences in the proportional limit values. This is quite evident in tensile stress-strain tests of wire where the specimens are simply full-size lengths cut from a coil of the material. In such tests the stress-strain curves almost invariably show very low proportional limits unless a severe straightening process is given the specimen, and in such event the proportional limit values are usually higher than those obtaining in the original material. As another example the stress-strain curve *B* shown in Fig. 1 was obtained from an accurately machined, threaded-end standard $\frac{1}{2}$-in. diameter A.S.T.M. tension test specimen of "17ST" aluminum alloy, which was bent slightly before testing. This curve may be compared to the one designated as *A* in the same figure, which was obtained from a similar test of an adjacent specimen of the same material, but in this case the specimen was not bent. The proportional limit of the first specimen (curve *B*) is a little more than one-half the value obtained for the second specimen while the yield point[1] values show a difference of only 2750 lb. per sq. in. and the tensile strengths were 64,440 and 64,430 lb. per sq. in., respectively.

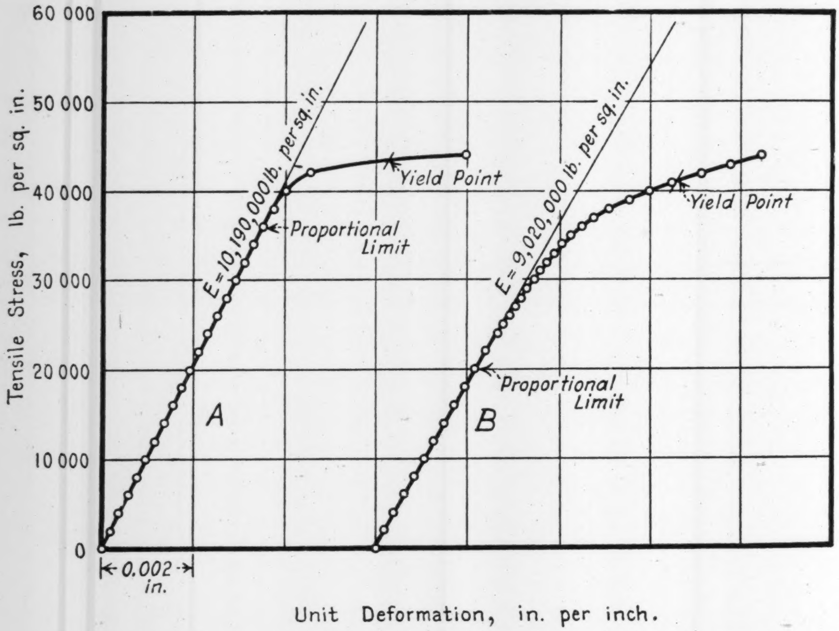

FIG. 1.—Tensile Stress-Strain Curves from Straight and Bent Specimens.

When tension specimens having reduced sections and enlarged ends are machined from a metal product, internal strains in the portion of the product represented by the specimen may be relieved or other strains may be induced in the specimen by the machining operation. That these effects are quite real in some products is evidenced by the warping of the specimens during machining. Further evidence may be found in the requirements of the Society's tentative standard to be followed in the preparation of tension test specimens from thin sheet metals. Again, it is not uncommon practice in the testing of hardened steels to first prepare tension specimens from the annealed or soft metal and then heat treat such specimens with the material which they represent, or separately as specimens using the same nominal heat treating conditions. In either case the specimens are quite liable to have strains induced in them different from those obtaining in the product they represent. The failure of attached or separately cast test specimens to give the same mechanical properties as the castings they are supposed to represent is now so generally recognized as to need no further comment at this time.

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[1] In this case, the stress at which the stress-strain curve shows a departure of 0.2 per cent from the initial modulus line produced; or approximately, the stress corresponding to a permanent set of 0.2 per cent.

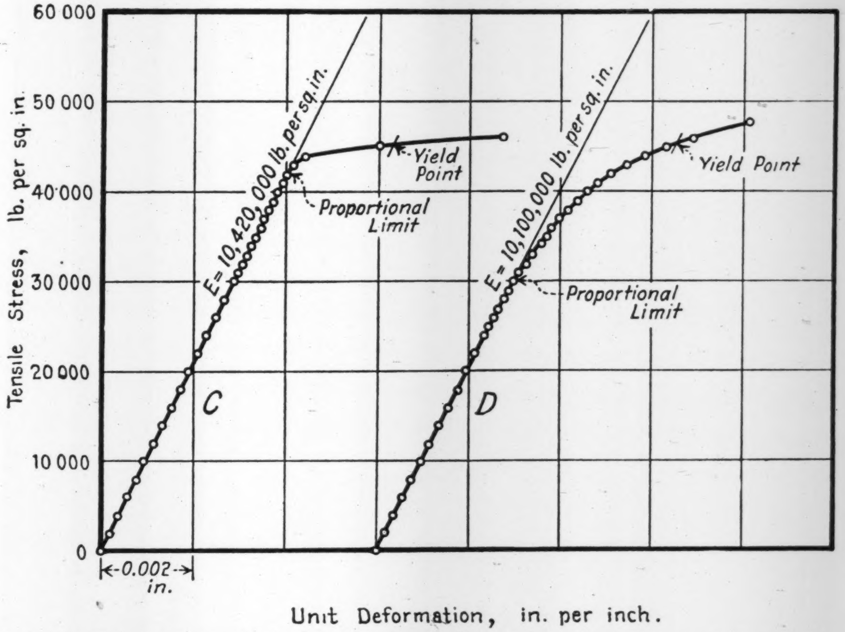

FIG. 2.—Tensile Stress-Strain Curves from Over-Strained and Peened Specimens.

As examples of how internal strain in specimens may affect the proportional limit values the curves *C* and *D* of Fig. 2 are shown. These curves were both obtained from tension stress-strain tests of specimens of the same dimensions and material previously indicated but the specimen of curve *C* had been given a slight over-strain before final test and the specimen of curve *D* was peened with a ball-peen hammer before machining to the final size. Comparing these two curves with curve *A* in Fig. 1 it may be noted that their proportional limits have been raised in the case of curve *C* 6000 lb. per sq. in. and lowered in the case of curve *D* 6000 lb. per sq. in. The yield point values for the specimens of curves *C* and *D* are 1700 and 1800 lb. per sq. in. higher than the value obtained for specimen of curve *A* and the ultimate strength values 1010 and 1280 lb. per sq. in. higher, respectively. It is admitted that both the treatments accorded the specimens of curves *C* and *D* are not in accordance with best testing practice but such procedure is not infrequently used, perhaps somewhat lesser in degree, in straightening bent test specimens.

The requirements for sensitivity and accuracy of testing machines and types of grips and supports for use in making tension and compression tests of metals are given in the standards and tentative standards of this Society. In some instances, however, these requirements are hardly adequate for accurate determination of proportional limit values of metals.

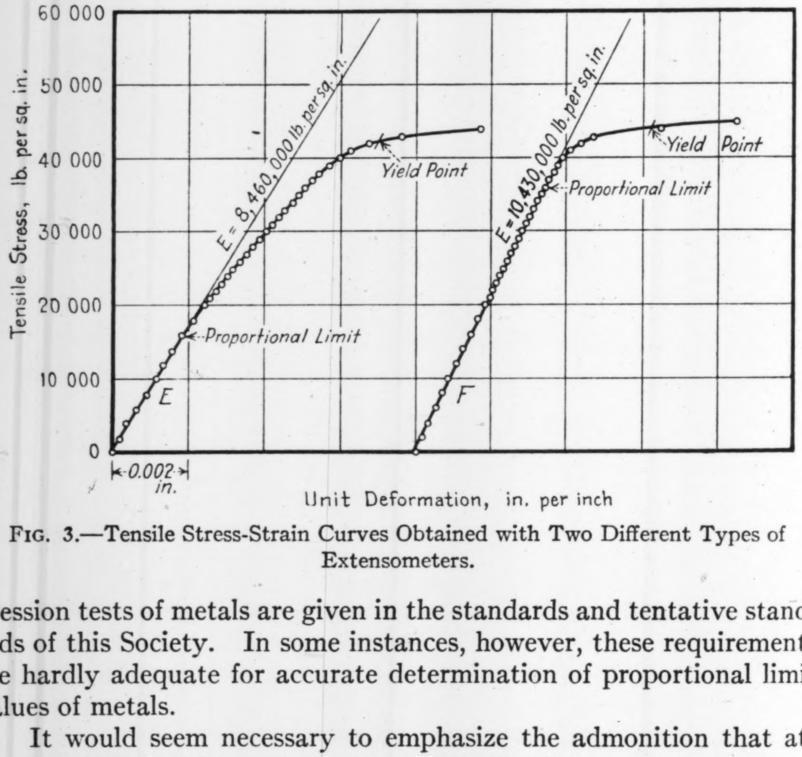

FIG. 3.—Tensile Stress-Strain Curves Obtained with Two Different Types of Extensometers.

It would seem necessary to emphasize the admonition that attempts should not be made to measure smaller increments or amounts of load than the size of the testing machine warrants. Furthermore, the use of just any kind of spherical seats or ball and socket joints for the grips and supports in making tension and compression tests of metals frequently causes unsatisfactory results. Some types of automatic load balancing devices now in use are quite unsatisfactory for use in the determination of the proportional limit values. The autographic attachments as furnished with commercial testing machines are more often than not deficient either in sensitivity or accuracy and must be used with considerable care if at all in the determination of the proportional limit.

The A.S.T.M. Tentative Methods of Tension Testing of Metallic Materials (E 8 – 27 T)[1] give requirements for the sensitivity of extensometers but fail to indicate requirements for accuracy. It should be recognized that many of the commercial extensometers give inaccurate readings, especially after being used for a considerable length of time. Such instruments therefore need calibrating rather frequently, and often adjustment as has been indicated previously before this Society.[2]

Curves *E* and *F* of Fig. 3 are stress-strain diagrams obtained from tension tests of specimens similar to that used in obtaining curve *A* in Fig. 1. In the present instances, however, the curves were obtained using a dial-gage type of strain gage and a dial-gage extensometer instead of the optical type extensometer used in obtaining curve *A*. A comparison of the curves *A* and *E* shows that the proportional limit value obtained in the first instance is about 20,000 lb. per sq. in. higher than obtained in the case of the specimen for *E*, whereas the yield points show a difference of only 1000 lb. per sq. in. and their ultimate strengths a difference of only 1510 lb. per sq. in. It may be noted furthermore, that the specimen represented by curve *A* had a proportional limit, yield point and ultimate strength of 1000, 350 and 740 lb. per sq. in., respectively, lower than similar values for the specimen represented by curve *F*. It should be emphasized that the testing machine, test procedure, specimen and material used in each of these tests were as nearly the same as possible, the only difference being in the extensometers used.

The testing procedure, that is, the method used in operating the testing machine and extensometer, may and frequently does affect the mechanical property values obtained. Of the various items that may be considered under this subject, speed of testing appears to be among the most important. Unfortunately, it is rather difficult with available commercial apparatus to obtain quantitative data showing the effects of speed of testing on the mechanical properties of a metal but we can readily get some idea of the effects by studying creep test results and comparing them with the usual short-time test results for the same materials. In general, such a comparison clearly shows that creep nearly always takes place at stresses appreciably below the proportional limit as determined by the short-time test. We may say then that the lower the speed of testing the lower will be the proportional limit of the material.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol 27, Part I, p. 1067 (1927); also 1928 Book of A.S.T.M. Tentative Standards, p. 808.

[2] R. L. Templin, "The Calibration of Extensometers," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 714 (1928).

Objections may be raised to the example just given because of the differences in speed normally obtaining in creep tests and short-time tests. It must be remembered, however, that comparatively wide variations in speed of testing are now permitted by the specifications of this Society, in tests of metals, and for such metals as zinc and pure magnesium the mechanical properties vary considerably with speed of testing within the range permitted. Depending on the metal, therefore, the speed of testing becomes of more or less consequence when determining mechanical property values, among which are values for the proportional limit.

Specifications indicating mechanical properties for metals, unless otherwise definitely stated, imply that the tests shall be made at or about room temperature. Experience has shown that actually the temperatures obtaining during such tests may range anywhere from a few degrees below zero to some above 100° F. The rather recent mechanical property data for metals at various temperatures, including creep test data, would seem to indicate for some metals, at least, that appreciable differences in test values might be expected throughout the range of temperature just indicated.

It is rather difficult to evaluate the personal equation involved in making mechanical property tests of metals, but in general, the more apparatus there is to be manipulated and the more readings there are to be taken the greater will be the chance of personal error. Likewise, as the degree of accuracy required in a measurement or observation is assigned closer limits, assuming the same apparatus in all cases, the greater will be the error in proportion to the given limits, due to the human equation. Since the proportional limit value determination obviously requires measurements of a higher degree of accuracy than the yield point value, for example, we would expect a greater variation due to personal equation in the proportional limit than in the yield point value.

After test data have been obtained, involving results which are affected more or less by the various factors just discussed, we are then confronted with the problem of properly interpreting such data. A very common way to determine the proportional limit of a metal is to plot the stress-strain data, draw a smooth curve through the plotted points and then attempt to determine the stress at which the curve ceases to be a straight line or departs from the initial slope of the curve. In such a procedure the choice of scales used for ordinates and abscissas is of paramount importance as can be seen from a study of the three curves shown in Fig. 4. The same data were used for each of these curves, but the abscissa scale in each case is different. The proportional limit values obtained are 24,000, 20,000 and 14,000 lb. per sq. in., respectively, while the yield point values are the same in each case. In Fig. 4, curve *B* is the same as curve *B* in Fig. 1.

Another method sometimes used in determining the proportional limit is to compare increments of strain resulting from constant increments of stress. The highest stress at which there is no increase in the strain increment is usually considered the proportional limit value. Such a procedure necessarily involves a careful consideration of accuracy and sensitivity of the extensometer or strain gage used, as well as of the errors of observation. Obviously, therefore, the proportional limit values obtained will be dependent to a large extent on the factors just mentioned. The drawing of the curve in the first method tends to minimize the errors just discussed but involves other errors due to draftsmanship.

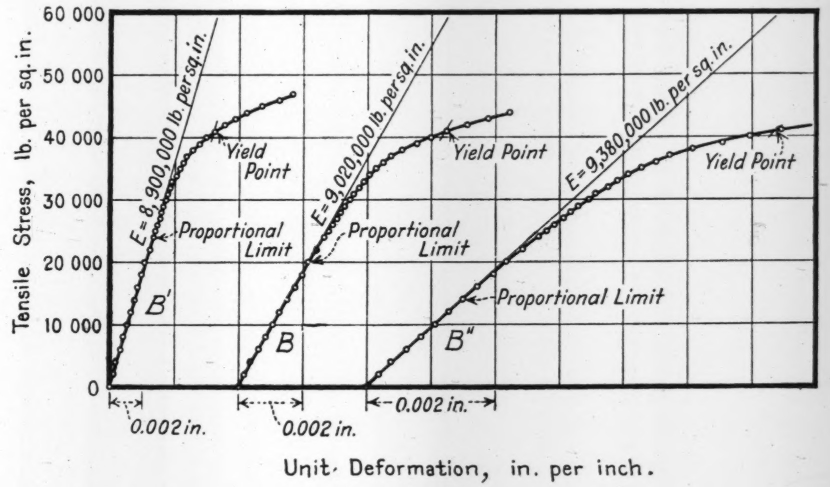

FIG. 4.—Tensile Stress-Strain Data Plotted Using Different Scales.

SIGNIFICANCE OF THE PROPORTIONAL LIMIT

As pointed out earlier in this discussion the proportional limit of a metal theoretically may be used as a criterion for determining the presence of certain defects in a metal. Practically, however, the proportional limit is affected to such an extent by factors not actually

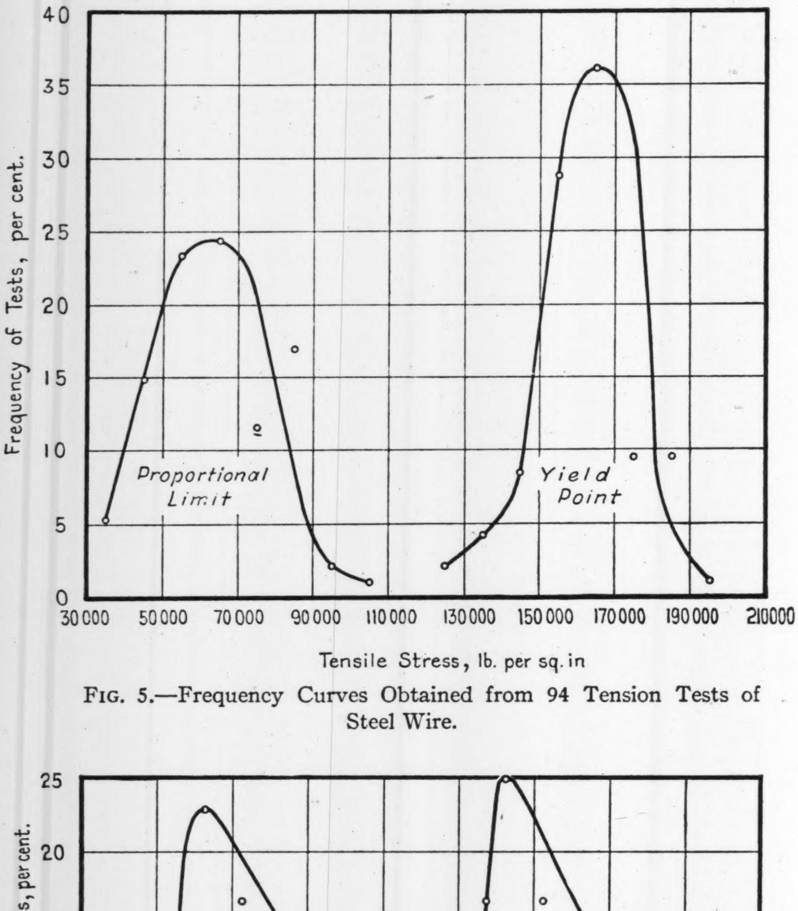

FIG. 5.—Frequency Curves Obtained from 94 Tension Tests of Steel Wire.

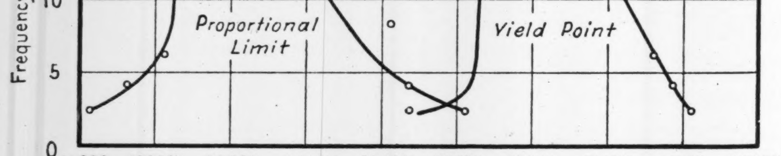

FIG. 6.—Frequency Curves Obtained from 306 Tension Tests of High-Strength Aluminum Alloy Products.

obtaining in the metal, that its use for this purpose may and probably does in many cases become quite questionable.

When the various factors affecting the determination of the proportional limit of a metal can be properly controlled, the results obtained should and do have considerable value as defining the limits of validity for the rigorous application of the mathematics of elasticity and mechanics involved in the design of metallic structures. The proportional limit values are, furthermore, of considerable use in development of alloys, fabrication processes, and heat treatments. In order that the proportional limit value may be of such use, however, it must be determined very carefully, using quite expensive and often elaborate apparatus. In addition, the test when properly carried out requires a considerable amount of time, usually much more than the other mechanical property tests, which of course means additional cost.

In view of the facts presented, the use of the proportional limit test, in the commercial routine inspection of metals involving large numbers of specimens, would appear inadvisable. On the other hand, a rather definite value for the proportional limit of a metal should be given the designer of structures to be made of that metal. Such a proportional limit value might well be the result of comparatively few tests properly made rather than an average value obtained from a great many tests made under commercial routine inspection conditions. Minimum or average values for the proportional limit, obtained from a large number of inspection laboratory tests, would undoubtedly be appreciably affected by the many factors just discussed which are not due to variations in the metal product represented by the specimens.

The frequency curves shown in Fig. 5 were obtained from 94 tests of high-strength galvanized steel wire and those shown in Fig. 6 were obtained from 306 tests of "17ST" high-strength aluminum alloy products. In both instances a study of the curves clearly shows the greater variation occurring in the proportional limit values. In view of the facts previously discussed it would seem reasonable to attribute by far the major portion of these variations in proportional limit to factors other than those inherent in the metals.

The relationship between the proportional limit value, as determined from test specimens taken from a metallic structural member, and the actual strength of an engineering structure made from such material, may vary between quite wide limits. As evidence of this your attention is directed to the conclusions given in a paper presented by Mr. Withey[1] before this Society last year. Mr. Withey found as a result of a considerable number of careful tests that "the variation of the proportional limit in different portions of a shape appeared to have little effect upon the compressive strength of whole prisms or the strength of columns built of the shapes." Again he found, "the weighted yield point in tension proved a good measure of the strength of the whole prisms tested in compression."

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[1] M. O. Withey, "Test of Specimens Cut from Different Portions of Structural Steel Shapes," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 41 (1928).

H. F. Moore[1] is of the opinion that the terms designated as "elastic limit," "proportional limit," or "yield point" are significant only in that they designate a practical limit of *elastic strength* and the differences between them are differences in test method rather than in their significance as a physical property. His solution of the problem presented involves a revision of our present nomenclature so as to provide a small arbitrary tolerance for the stress at which inelastic action ceases. It would seem, however, that the question of nomenclature should properly come secondary to the problem of determining just what is the mechanical property value that will be of most use in permitting a designer to predict the strength or performance of a structure made from a given material. Many designers will certainly agree that the yield point value has played a very important part in the design of large numbers of our present-day structures.

Irrespective of whether the yield point or the proportional limit is used as a basis for the selection of design stresses and assuming that there is real need for some such mechanical property value in the testing of metals purchased under mechanical property specifications, it would appear rather obvious that the value selected should involve minimum ranges of variation due to the factors previously discussed, and which are not inherent in the material itself. This being so, it would seem that the so-called yield point meets these requirements better than the proportional limit.

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[1] H. F. Moore, Discussion of a paper by J. M. Lessells on "Concerning the Yield Point in Tension," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 394 (1928).

DISCUSSION

MR. H. F. MOORE[1] (*presented in written form*).—Mr. Templin has presented a significant and valuable paper. He has shown clearly many of the reasons for the wide range of values obtained by different experimenters for proportional limit of a given metal. For some years it has seemed to the writer that the attempt to locate precisely the geometrical point at which a flat curve merged into a straight line has small chance of success. In Mr. Templin's Fig. 2, specimen D, for example, it would seem to the writer that about the only mathematically precise statement which could be made about it would be that the proportional limit was not higher than 30,000 lb. per sq. in., and that it might have any value between that and zero.

In considering the proportional limit as an index of a desirable strength quality in a material it should not be forgotten that a material stretched above its proportional limit and below its yield point will not be seriously deformed and will have its proportional limit raised for successive applications of load. The writer is very doubtful of there being any particular strength significance to a proportional limit such as shown in Fig. 3, specimen E, of Mr. Templin's paper.

Again taking Fig. 2, specimen D as an example, at a stress of 40,000 lb. per sq. in. the unit deformation of the metal is only about 0.0046 in. per inch, not in itself a serious matter, and under successive loadings this will not increase very much. Practically, elastic failure has not been developed. If subjected to thousands of loadings, the slight bend of the curve is no evidence that a fatigue crack has been started. What practical difference in significance is there between the value marked "proportional limit" and the value marked "yield point" except the quantitative difference that the latter marks the beginning of the inelastic action as determined by a rather crude method, while the former marks the same thing as determined by a fairly delicate method.

Mr. Templin notes the necessity for the calibration of extensometers used to determine the proportional limit. There is this need, but it is a need rather for the determination that the extensometer readings are *proportional* to the actual stretch in inches rather than that they *read correctly* the actual stretch in inches. In determining modulus of elasticity, it is highly essential that the readings of the extensometer should be correct. In determining proportional limit, it is important that the readings of the extensometer should be *proportional* to the stretch in inches.

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[1] Professor of Engineering Materials, University of Illinois, Urbana, Ill.

Mr. Templin's final conclusion that the "so-called yield point meets these requirements" would involve a redefinition of yield point. As Mr. Templin has indicated, the writer does believe that the terms designated as "elastic limit," "proportional limit," and "yield point" are names for various arbitrary measures of elastic strength. The present tentative definition of yield point[1] is "that stress in a material at which there occurs a marked increase in strain *without an* increase in stress." According to this definition only a few metals have a yield point. For purposes of discussion, the writer suggests the following definition and methods of determination of yield point:

> *Yield Point.*—The stress in a material at which the beginning of appreciable inelastic action is indicated by a well-marked increase of the rate of strain with respect to stress.

To the user of materials and the designer of machines or structures, the significance of the yield point is that of a limiting stress below which the permanent distortion of a material is so small that the structural damage is negligible, and above which the damage is not negligible. It is evident that the amount of distortion which is regarded as negligible varies widely for different materials and different fields of service, and also that any determination of yield point for any given material involves arbitrary criteria and methods. It is not intended to specify here definite methods for all materials, but proposed general methods of determining yield point will be indicated.

Methods for Determining Yield Point Without a Stress-strain Diagram.—For some metals, and possibly a few non-metals, the stress-strain diagram, when plotted, shows a region in which the slope changes very rapidly as indicated at *pq* in the accompanying Fig. 1. For such materials the yield point may be determined without the necessity of taking data for the plotting of a stress-strain diagram for each test specimen, either by employing the "drop of the beam" as described in the Tentative Definitions of Terms Relating to Methods of Testing (E 6 - 25 T)[1] or by the Total Deformation Method as follows: Materials whose stress-strain diagrams show a sharp break in the yield region have a considerable range of strain for which the stress varies but slightly (*qr*, Fig. 1). If, as the test proceeds, the load is noted which corresponds to a given deformation (*Oa*, Fig. 1) within this range, there is located a limiting stress which will serve satisfactorily as a yield point value, below which permanent distortion is slight and above which it is appreciable. For any given material having a "sharp-knee" stress-strain diagram, this critical strain may be arbitrarily fixed, and its value, in terms of percentage of the original length, may be stated in specifications, and denoted by "*e*."

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[1] Tentative Definitions of Terms Relating to Methods of Testing (E 6 - 25 T), *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part I, p. 879 (1925); also 1928 Book of A.S.T.M. Tentative Standards, p. 827.

In reporting values of yield point for materials of this class the limiting value of *e* used should be placed in parenthesis after the term "yield point," or if yield point was determined by the drop of beam method, the word "drop" should be so used. Thus:

Yield Point (e = 0.5 per cent) = 42,000 lb. per sq. in.

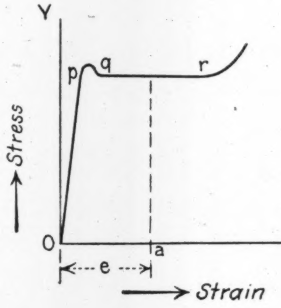

FIG. 1.—Stress-Strain Relations for Materials Showing Sharp Break in Yield Region.

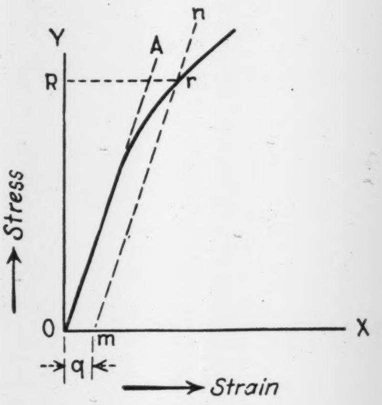

FIG. 2.—Stress-Strain Relations for Materials Showing No Sharp Break in Yield Region.

would indicate that at a stress of 42,000 lb. per sq. in. the strain of a specimen of a material reached the value of 0.5 per cent of the original length.

Yield Point (drop) = 37,000 lb. per sq. in.

would indicate that at a load corresponding to a stress of 37,000 lb. per sq. in. a well-marked drop of beam was observed.

Measuring apparatus used to determine *e* should be of sufficient sensitivity to measure the limiting strain accurately.

On a tension test specimen with a gage length of 8 in. and a value of *e* of 0.5 per cent, the amount of stretch at yield point would be 0.04 in., and ordinary dividers, if carefully used, would be satisfactory. On a compression test specimen with a gage length of 2 in. and a limiting value of *e* of 0.25 per cent the amount of compression at the yield point would be 0.005 in. and an extensometer reading to 0.0005 in. would be desirable.

Method for Determining Yield Point, Using a Stress-strain Diagram.—For materials whose stress-strain diagram when plotted does not show a "sharp-knee" in the yield range, the methods outlined in the foregoing paragraphs are not satisfactory for determining yield point. The method next outlined can be used for materials having "sharp-kneed" stress-strain diagrams, and also for some materials whose stress-strain diagram is a smooth curve.

For nearly all materials, if at any point on the stress-strain diagram such as *r*, on the accompanying Fig. 2, the load is released the diagram for decreasing load will follow a line, *rm*, approximately parallel to the initial portion, *OA*, of the diagram for increasing load. *Om* will then give the approximate value of the permanent set after the release of the stress *OR*. Denote the value of *Om* by "*q*," which shall locate the yield point. The value of *q* is given in percentage of original length. Then to determine yield point by the total deformation method, it is necessary to secure data (autographic or numerical) from which a stress-strain diagram may be drawn. Then with the stress-strain diagram drawn (Fig. 2) lay off *OA*, giving the initial slope of the diagram, lay off *Om* equal to the arbitrary specified value of *q*, draw *mn* parallel to *OA* and locate *r*, the intersection of *mn* with the stress-strain diagram. Draw *Rr* parallel to the *X* axis and then *OR* gives the value of the yield point.

In reporting values of yield point obtained by this method, the arbitrary value of *q* used should be stated in parenthesis after the term yield point. Thus:

Yield Point (q = 0.2 per cent) = 52,000 lb. per sq. in.

indicates that at a stress of 52,000 lb. per sq. in, the approximate permanent set of the material reached the value of 0.2 per cent of the original length.

In using this method, an extensometer reading to 0.0001 in. per inch of gage length would be sufficiently sensitive for most materials. Values of *e* and of *q* to locate the yield point of different materials would be determined by the committees or individuals preparing the specifications for the particular material.

MR. L. B. TUCKERMAN[1] (*presented in written form*).—Mr. Templin has presented the case against the use of the proportional limit in routine commercial tests well and convincingly. His thorough analysis, based upon his extensive experience with precision testing, covers the ground so thoroughly that little remains to be said. However, his discussion of the interpretation of the results obtained may perhaps be emphasized by a somewhat different manner of presentation.

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[1] Assistant Chief, Division of Mechanics and Sound, U. S. Bureau of Standards, Washington, D. C.

In Fig. 4 of the paper he plots the same set of data on three different scales and shows how the scale of plotting affects the values he obtains for Young's modulus of elasticity, E, and for the proportional limit. These differences are typical of what would be found from the great majority of tensile tests.

At the Bureau of Standards we are from time to time requested to report values of the proportional limit. Every time we have reported such a value I have had a twinge of conscience caused by the realization that the value so reported was, as Mr. Templin has so clearly pointed out, not, strictly speaking, a characteristic of the material but some hazy composite of material properties, testing technique, method of plotting and personal equation. These twinges of conscience have been only partially relieved by including in the report a statement of the extensometer used, a stress-strain diagram on a fairly standard scale ("straight" portion making approximately 60 deg. with the axis of abscissas) and indicating on this diagram the point at which the individual making the test judged that the curve "began to depart from a straight line."

We have, it is true, had the comfort of knowing from repeated checks that the personal equation of the Engineering Mechanics Section of the Bureau of Standards was fairly well standardized so that there was no danger of two men taking the same data and reporting widely divergent proportional limits. As a consequence, we have felt sure that significant differences (say 10 per cent) in the values of the proportional limit of materials determined in our laboratory represented real differences in the properties of the material tested. We have not been at all sure that the values we obtained would bear any definable relation to the values obtained in another laboratory. This has led me to try all sorts of schemes in an effort to free the determination of the proportional limit as much as possible from the influence of this personal equation.

The method finally adopted is an adaptation of one well known to physicists for many years. Where it is found that observed data follow approximately some definite mathematical law and it is desired to study the deviations from that law, values accurately obeying that law are computed and the differences between the observed and computed values are plotted. When these differences are small, they can be plotted to an exaggerated scale, so that they can be really studied instead of merely being guessed at.

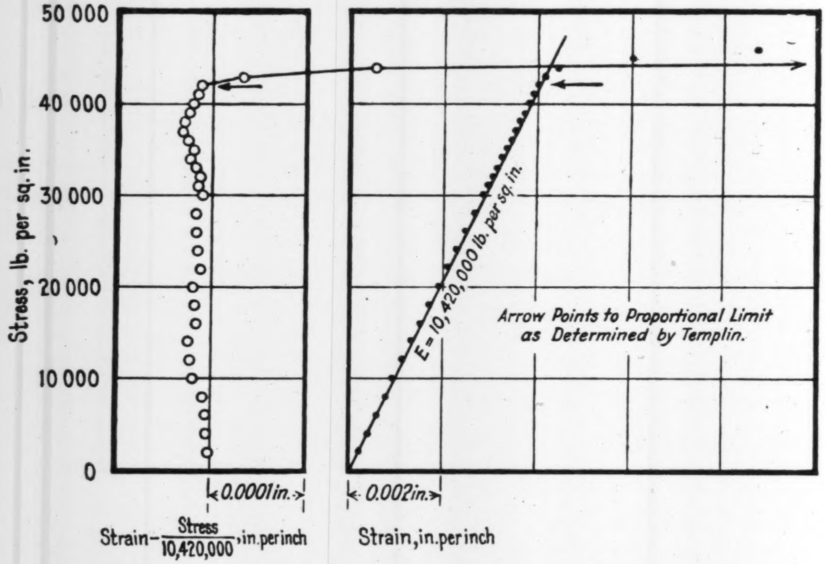

FIG. 3.—Stress-Strain Relations Using Data from Templin's Curve *C*.

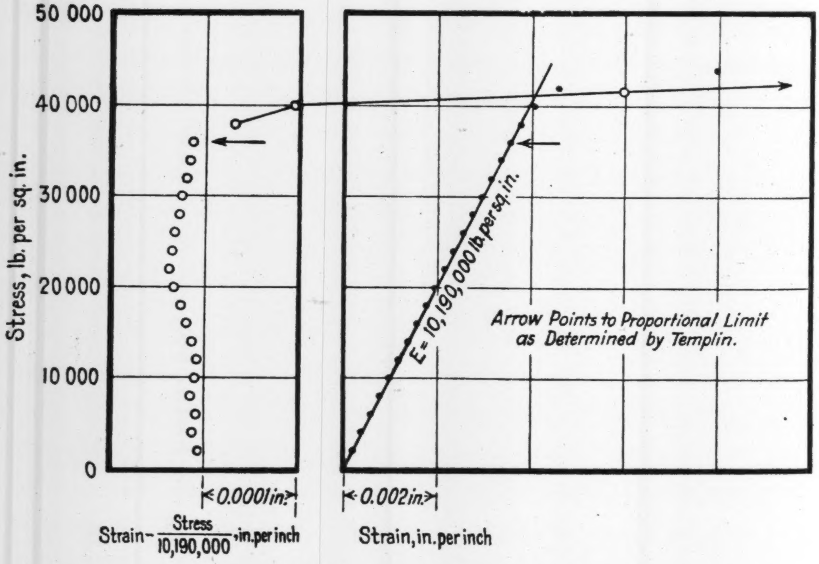

FIG. 4.—Stress-Strain Relations Using Data from Templin's Curve *A*.

Thus in a stress-strain diagram it is known that the first portion follows approximately the law:

$$\text{Strain} = \frac{\text{stress}}{\text{modulus of elasticity}}$$

Any straight line which fits reasonably closely this "straight line portion" of the curve is chosen and from it a trial modulus, a hypothetical straight line strain, is computed and the difference

$$\text{Strain} - \frac{\text{stress}}{\text{modulus of elasticity}}$$

is plotted as abscissa to a large scale with stress as ordinate. It is usually sufficient to carry this computation one significant figure beyond the extensometer readings.[1]

Mr. Templin has been kind enough to send me copies of the original data sheets from which he plotted the curves of his Figs. 1 to 4, and I have replotted them in the manner shown in the accompanying Figs. 3 to 8. For these curves I have used Templin's modulus as the trial modulus. In the difference curves the scale of abscissas is purposely made so large that the deviations from a straight line will stand out plainly. The diameter of the circles is made to represent the least count of the extensometer, 0.00001 in. per inch. The apparent "jumps" in curve *C*, Fig. 3, 0.00001 in. between 8000 and 10,000, 14,000 and 16,000, and between 20,000 and 22,000 lb. per sq. in. are of course obviously not real jumps in the stress-strain diagram but are caused by the fact that 0.00001 in. per inch is the least difference which can be read on the extensometer. It is reasonable to assume that somewhere in the midst of these points there lies the "true" stress-strain curve of the material and that up to 42,000 lb. per sq. in. it does not differ widely from a straight line. Curve *A* (Fig. 4) does not look quite so good. It is difficult to reconcile the trend from 0 to 22,000 lb. per sq. in. and that from 22,000 to 36,000 lb. per sq. in. with any one straight line. Some of the observed points must lie at least 0.00002 in. per inch from any straight line which can be drawn among them. Curve *D* (Fig. 5) does not require further comment. Curve *B* (Fig. 6), however, exhibits a phenomenon well known to all who have tried to make accurate extensometer readings. The first two observations (2000 and 4000 lb. per sq. in.) point altogether in the wrong direction, the difference in trend below and above 4000 lb. per sq. in. being far greater than can be accounted for by

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[1] An alternative method is to plot: stress − trial modulus of elasticity × strain. This has the disadvantage that the abscissas are not directly expressed in terms of the extensometer readings. If the error in the load indicated by the testing machine is small, it is usually found that the errors of the extensometer limit the accuracy of the results.

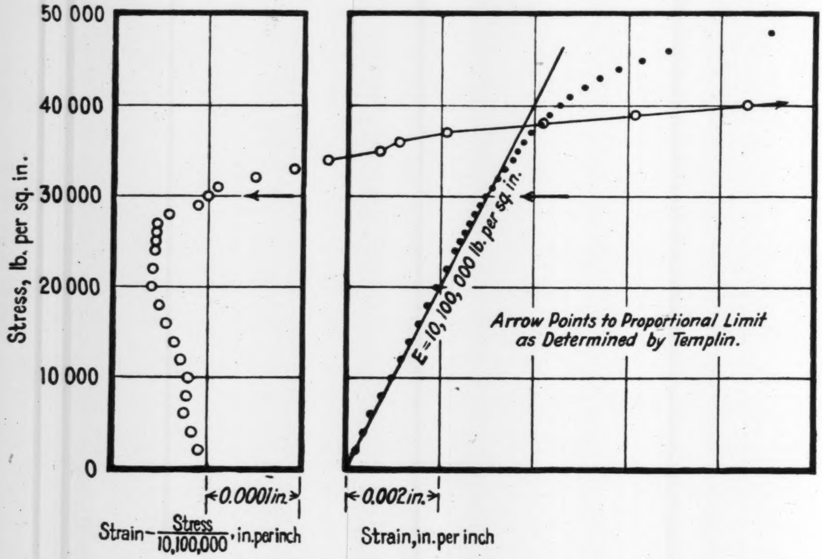

FIG. 5.—Stress-Strain Relations Using Data from Templin's Curve *D*.

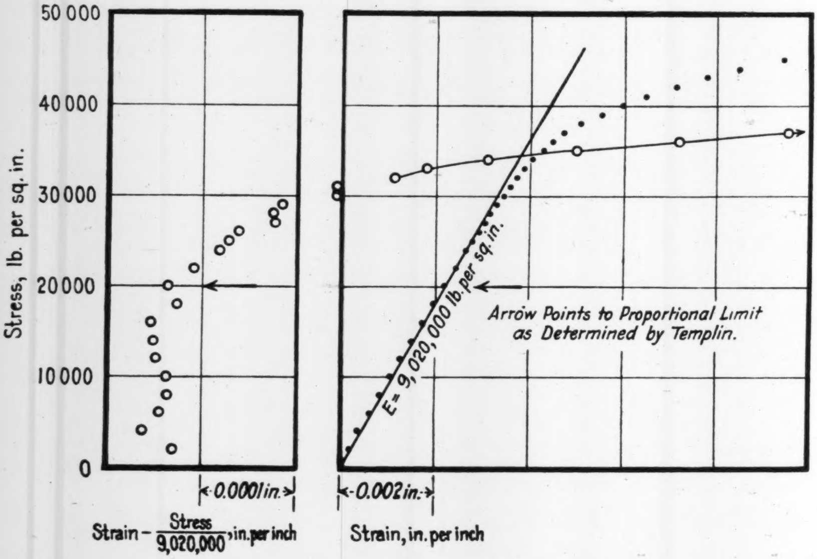

FIG. 6.—Stress-Strain Relations Using Data from Templin's Curve *B*.

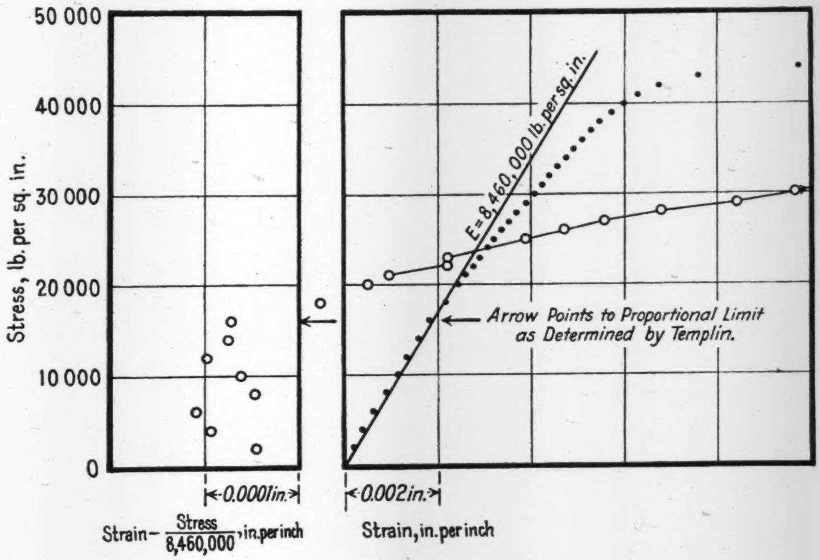

FIG. 7.—Stress-Strain Relations Using Data from Templin's Curve *E*.

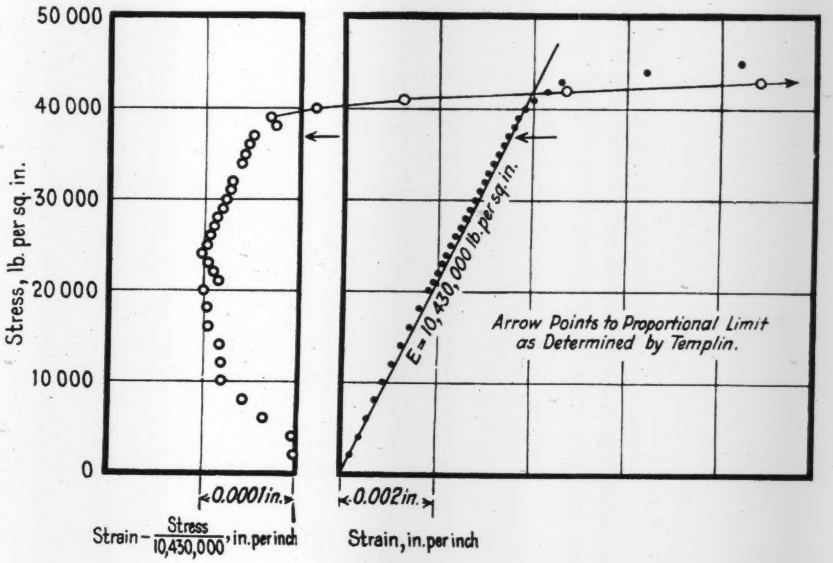

FIG. 8.—Stress-Strain Relations Using Data from Templin's Curve *F*.

any error in the extensometer itself. It is usual to interpret this as caused by a failure of the extensometer to "seat" properly on the specimen. In spite of the greatest care this will sometimes happen. If particular importance is attached to low readings, dropping the load to zero and repeating will usually "seat" it properly.

Curves *A*, *B*, *C*, and *D* (Figs. 3 to 6) are typical of what may be expected from the highest grade of extensometer, such as the Ewing, in the hands of careful operators. Just where these curves are judged to "begin to depart from a staight line" quite clearly depends much on the personal equation.

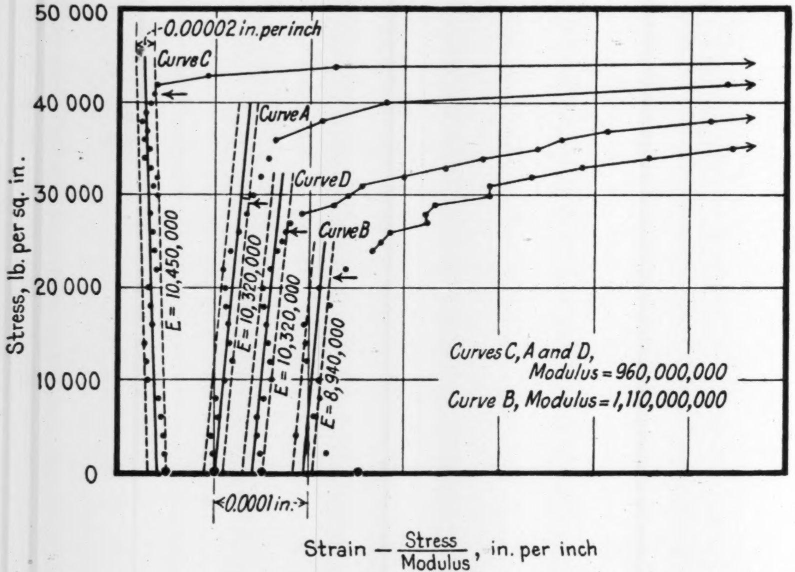

FIG. 9.—Determination of Young's Modulus and Proportional Limit by Plotting Differences from a Straight Line and Using the Least Count of the Extensometer as a Criterion. Data from Templin's Curves *C*, *A*, *D* and *B*.

Mr. Templin no doubt feels that the personal equation of the testing laboratory of the Aluminum Co. of America has become sufficiently standardized so that significant differences (say 10 per cent) in the values of the proportional limit of material, determined in his laboratory, represent real differences in the properties of the material tested. However, I think that he feels as doubtful as I do whether the Engineering Mechanics Section at the Bureau of Standards, or any other testing laboratory, would report "proportional limits" for widely different materials even reasonably comparable with those his laboratory would report from the same numerical data.

Curve *E* (Fig. 7) and curve *F* (Fig. 8) as replotted show, I think, somewhat more clearly than Mr. Templin's Fig. 3 how wholly indeterminate a "proportional limit" obtained by the use of the more usual dial gage extensometers must be.

I think these curves have shown that even when the sensitivity and accuracy of the extensometer are given and the testing technique is excellent some more definite criterion than a "standardized personal equation" is necessary if the proportional limits reported by different laboratories are to be reasonably comparable.

The best method I have found (and it is far from satisfactory) is to use the least count of the extensometer as the criterion for "departure from a straight line." Fig. 9 represents the method of applying this criterion to the data of curves *A*, *B*, *C*, and *D*. The differences from a straight line are computed and plotted exactly as before, except that for convenience the trial modulus is chosen so that stress divided by modulus of elasticity comes out even in the next significant figure beyond the least count of the extensometer. Three lines are then drawn on a transparent sheet of paper, the spacing between them being made to represent the least count of the extensometer. These lines are then shifted over the plot until as many as possible of the observed points in the approximately vertical portion of the curve are brought to lie between the two outside lines. Where, as in curve *B*, the divergence of the lowest points seems to be reasonably attributable to "imperfect seating" of the extensometer, these points are ignored.

It is evident that for all points between the two outside lines we have no evidence, in the measurements taken, that the "true" stress-strain curve departs from the middle straight line. Where, however, the points fall outside of the two outer lines, provided the loading errors and other accidental errors are negligible in comparison with the least count of the extensometer, we have definite evidence that the "true" stress-strain curve cannot be the middle straight line.

It seems reasonable, then, to determine the modulus of elasticity for the material by correcting the trial modulus for the slope of these lines, and to consider that the proportional limit, as determined by the measurements with this extensometer, is located at the stress at which a straight line drawn between two consecutive observed points cuts the right-hand outer line.

There is left still, in this method, from the standpoint of perfect determinateness, an uncomfortable amount of personal equation.

In curve *B* (Fig. 9), for instance, should we interpret the slight excursion of the points at 8000 and 16,000 lb. per sq. in. stress beyond the outside lines as caused by loading errors and other accidental errors or should we hold strictly to the criterion, readjust the transparent sheet, and call the proportional limit 16,000 lb. per sq. in.?

It may be of interest to compare the values found by this method with the values found by Mr. Templin, as given in the accompanying Table I.

For curve *C*, the values do not differ significantly. It is characteristic of materials which have been previously over-strained to show this fairly definite proportional limit, reasonably independent not only of personal equation, but also to a considerable degree independent of the sensitiveness of the extensometer. For the other three the differences are appreciable.

TABLE I.—COMPARISON OF YOUNG'S MODULUS AND PROPORTIONAL LIMIT DETERMINED FROM SAME DATA BY DIFFERENT METHODS.

Specimen Number from Templin	Young's Modulus, *E*, lb. per sq. in.		Proportional Limit, lb. per sq. in.	
	Difference from Straight Line	Templin	Difference from Straight Line	Templin
A	10 320 000	10 190 000	29 000	36 000
B	8 940 000	9 020 000	21 000	20 000
B′	....	8 900 000	......	24 000
B″	....	9 380 000	......	14 000
C	10 450 000	10 420 000	41 000	42 000
D	10 320 000	10 100 000	26 000	30 000

It is quite clear that this method of determination of modulus and of proportional limit is not adapted to routine testing. In the Engineering Mechanics Section of the Bureau of Standard we do not use it for that purpose. I am content to salve my conscience by an occasional check to see that the "standardized personal equation" of our engineers gives proportional limits differing not too widely from those found by this method.

Mr. Templin deserves much credit for pointing out so clearly how inadvisable from all viewpoints it is to include the "proportional limit" among the properties specified for determination in routine commercial testing. I hope that this somewhat lengthy discussion of one phase of his paper may serve to make it even more clear.

MR. J. A. CAPP[1] (*presented in written form*).—It seems to me that Mr. Templin's paper indicates what might be called a weakness in our methods of testing. We are inclined to try to use the same sort of yard stick for all sorts of measurements. It so happens that nature has been fairly kind to the man who tests steel in providing a jog in the stress-strain curve just beyond the nominally straight line portion. Usually the jog is unmistakable and it bears a sufficiently close relation to the end of the straight line portion of the stress-strain curve to be acceptable in commercial testing as a datum point for the design engineer. Nature has not been so kind in the case of the non-ferrous metals in that she has not provided such a jog. Everyone testing the non-ferrous metals has earnestly looked for something which can be determined as quickly and easily as we determine the yield point of steel and which will have an equivalent meaning. Since nature has provided no jog, the nominally equivalent point has necessarily had to be established by convention. The conventional point has customarily been set at the load producing 0.5 per cent extension without taking into account the differences among the different materials that are tested and without considering whether they are nearly enough alike in their stress-strain behavior to permit the use of the same conventional point for all of them. In some of Mr. Templin's curves he has shown the yield point, as he calls it, at a figure widely different from that which he designates as the proportional limit. The two obviously have no necessary connection.

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[1] Engineer, Testing Laboratory, General Electric Co., Schenectady, N. Y.

In others of his curves the indicated yield point is as close to the indicated proportional limit as the two values commonly appear in steel. With this sort of data, the question is inevitable, "Why should we attempt to apply the same criterion to all metals?"

There is another point brought out in the paper. The author speaks of commercial testing. While we more or less hint at it in some of the methods developed by Committee E-1 on Methods of Testing, should we not frankly realize and state that we test for at least two rather separate and relatively distinct purposes? To determine the intrinsic properties of a material, we test with the utmost care using the finest instruments available. No one seems to object to the time and money spent in making these careful exact tests for basic information. But we also test purely commercially to determine within the commercial limits set in specifications whether the material is of the commercially established grade. We recognize necessary variability in commercial material and set limits accordingly. Should we not set up definitely commercial methods of testing which do not call for the same careful meticulous accuracy that is necessary for information testing? Here, obviously, it becomes necessary to recognize that differences in the technique of testing these different materials for commercial purposes might well be justified and acceptable. For example, some materials are not particularly susceptible to change of properties by manipulation or treatment as can be done with most of the steels. Would it not be sufficient to apply the information-yielding methods to determine the intrinsic properties of such materials showing the nearly fixed relations of elastic and ultimate strength, and showing the permitted variations in ultimate strength with good practice, then will it not be sufficient for commercial testing to determine only the ultimate strength, knowing that the ratio of elastic to ultimate strength is practically fixed? When the ultimate strength falls within the accepted range and the other figures such as ductility, etc., are right, surely the elastic ratio should be satisfactory. Then in the commercial testing, there is no necessity for making the more or less difficult determination of elastic limit and certainly there can be no advantage in setting up a purely conventional point which intrinsically has no meaning.

When a conventional point to be considered the equivalent of the yield point in iron and steel is needed, should not this conventional point be set for each type of material so that it has some meaning instead of our continuing to use one conventional fixed point which has been shown to have no real meaning except for perhaps one or two materials?

It seems to me that it should be the business of our committees, and particularly of Committee E-1, not to hint at, but definitely to set up methods of testing both for meticulous testing and for commercial testing just as has been done in some of the methods of chemical analysis. The committee should record the application of the methods for their different purposes and in the case of the commercial methods, there would probably have to be several conventions set up for application to the different materials according to their behavior under stress. We should not record "yield point" as determined by these conventional methods but should give it a name of its own. Surely such a procedure would be more satisfactory than the present one of setting down a figure which really has no meaning whatsoever.

MR. J. B. KOMMERS[1] (*by letter*).—This paper is interesting and valuable because it gives a quantitative measure of the effect of some of the factors which influence the determination of the proportional elastic limit.

It has always seemed doubtful to me whether there is any value in attempting to determine a yield point for any metal which has a stress deformation curve like *B* of Fig. 1 of the paper. For the ferrous metals and alloys which have a real yield point, and in which the yield point is only slightly higher than the elastic limit, it is worth while to know what the yield point is. For many non-ferrous metals there is no real yield point, and to fix any arbitrary method for getting something which has no particular meaning after you get it seems of doubtful value.

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[1] Professor of Mechanics, Engineering College, University of Wisconsin, Madison, Wis.

I agree with Mr. Templin that when the proportional elastic limit is determined it should be done carefully, and for that reason and because of the expense of making the test, it should not be required as a routine test result.

Whether the proportional elastic limit is important depends to a large extent upon the type of structure in which the material is to be used. If it is important that a device or a structure maintain its original dimensions indefinitely it would be highly desirable to know the proportional elastic limit. On the other hand, if slight changes in dimensions are permissible, it would probably be sufficient if the ultimate strength were known.

I believe the term yield point should be discarded for those metals whose stress-deformation curves gradually bend to the right for increasing stresses, and which therefore have no real yield point in the sense that many ferrous metals have.

MR. P. G. MCVETTY[1] (*presented in written form*).—It is indeed gratifying to note the growing interest in the more careful analysis of the fundamentals underlying the ordinary tension test. Mr. Templin has shown in a very convincing manner some of the important factors which are too commonly overlooked in ordinary routine tension tests.

Within the past few years a great deal of attention has been given to the determination of proportional limit. In some cases, consideration has been given to the substitution of the proportional limit value for the yield point as the basis for fixing safe working stresses for static loading. Under some conditions such a procedure may be desirable, but the fact remains that it is practically impossible for many testing laboratories to determine reasonably accurate proportional limit values in routine tests. An attempt to obtain such values without using the time and equipment necessary for accurate results leads to data that may be quite misleading.

The writer is somewhat surprised to find no reference to Mr. Moore's Presidential Address on "Hooke's Law of Stress and Strain"[2] presented before this Society a year ago. It is pertinent to inquire into the standing of the term "proportional limit" when, as Mr. Moore says, "No one to-day denies the approximate nature of Hooke's law." In spite of the fact that our whole mathematical theory of elasticity is based upon the infallibility of Hooke's law and the assumption that materials are homogeneous and isotropic, some allowance must be made for the probable inaccuracy of these assumptions. If it were possible to detect with certainty the first evidence of plastic deformation, it is extremely doubtful if we could measure the stress which produced this local deformation. In other words, the term "proportional limit," if rigorously applied, refers to a stress which cannot be measured by any means now known and which marks the so-called limit of a proportionality which does not exist. Under these conditions, it is to be expected that the values obtained will depend greatly upon the condition of the material and the method of test.

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[1] Mechanical Engineer, Research Department, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

[2] *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 38 (1928).

When allowance is made for uncontrollable variables and suitable precautions are taken in making the test, the apparent proportional limit becomes extremely valuable. Comparative tests under carefully controlled conditions give very useful data especially at elevated temperatures. The difficulties encountered in making such tests, however, usually confine them to special research investigations.

The yield point definition given by Mr. Templin has been used in the Westinghouse Research Laboratory for several years.[1] It has been found to be a very good basis for comparing materials which do not have a well-defined yield point such as is associated with the "drop of the beam." We have found also that it is usually advantageous to base working stresses for static loading at normal temperature upon a yield point rather than a proportional limit value.

MR. M. O. WITHEY[2] (*presented in written form*).—Considerations of durability, elasticity, and permanency of form dictate that certain strain limitations cannot be exceeded in a given part without materially lessening the structural value of such part. From the standpoint of the engineer and designer, the stress intensities corresponding to these strain limits are more usable than the strains themselves. Hence we have as stress indices proportional limit, elastic limit, and yield point, each of which corresponds to certain more or less well-defined limitations in strain. Which one of these stress limits should be specified in purchasing a metal for a specific part is dependent upon a number of factors, such as, the character of the metal, the importance of the part, the number required, the loading, the external conditions, weight limitations, and cost.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, p. 150 (1924).

[2] Professor of Mechanics, University of Wisconsin, Madison, Wis.

Mr. Templin's interesting and valuable paper calls attention to certain difficulties in the determination of the proportional limit and suggests that the yield point is a more reliable index for the purchase of materials under mechanical property specification. The author's examples illustrating the effects of initial strain in reducing the proportional limit also serve to show that the yield point may be masked by the same condition *unless the yield point is based upon a given unit strain.* In Figs. 1 to 4 of the paper, the yield point appears to have corresponded to a unit strain of 0.0063 in. per inch. Similar evidence with regard to lowering of proportional limit was obtained in tests of many specimens from the roots and toes of channels[1] and roots of H-columns.[1] Evidence of the masking of the yield point in tests of root specimens from two I-beams and an angle are also recorded.[2] The author's illustrations showing the influence of type of extensometer and magnitude of coordinate scales in determining proportional limit are timely, but discrepancies due to such differences in testing technique can be largely eliminated by making suitable specification for apparatus and procedure in testing.

Another detail which should receive careful attention is the centering of the extensometer on the test specimen so that the axial strain may be determined. In a specimen held by serrated grips, the difference in strain on the opposite sides is often in excess of 25 per cent. If the above stress difference exists on a specimen $\frac{1}{4}$ in. thick and the extensometer is 0.01 in. out of center, the registered strain deviates at least one per cent from the axial strain.

It appears to me that the most important information which the designer can obtain from proportional limit or yield point values is the intensity of stress which the material can carry without appreciable permanent change in shape. Whether he should be satisfied with yield point or demand the more exacting determination of proportional limit will depend upon his judgment concerning the factors previously mentioned. For specimens exhibiting poorly-defined yield points, arbitrary specification limits may well be imposed regarding the minimum acceptable intensity of stress corresponding to a given unit strain. For example, in tension tests of structural carbon steels, we might well demand that suitable multiplying dividers be used during the test and that whenever the drop of beam indication is uncertain, the intensity of stress corresponding to a unit strain of 0.2 per cent be taken as the yield point. With present specifications, this would mean that the specimen must carry 30,000 lb. per sq. in. without suffering strain in excess of 0.002 in. per inch. For the aluminum alloys of this paper, other limits of stress and strain would be appropriate.

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[1] M. O. Withey, "Tests of Specimens Cut from Different Portions of Structural Steel Shapes," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 41 (1928).

[2] Report of Committee on Yield Point of Structural Steel, *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 121 (1928).

MR. F. N. MENEFEE[1] (*presented in written form*).—Mr. Templin's paper, in addition to being an exposition of the various factors entering into a determination of the elastic limit and yield point, incidentally brings to mind the gradual narrowing down during the past fifteen years of nomenclature and variation in procedure used in discussing and making of physical tests. For instance, the difference between elastic limit and yield point is approaching some definite and fixed percentage of elongation. Whether a fixed percentage is in accord with nature's laws for all materials, is questionable, and whether it is the same for the same material subjected to different heat or mechanical treatment is questionable, but for practical purposes the more of such arbitrary standards we can agree upon, the more accurately we can correlate and interpret results.

At the outset it seems reasonable to ask why not go a step further and agree upon the use of one or the other of the terms "elastic limit" and "proportional limit." There has been a great deal of discussion of these terms and probably will be more, but it seems that the term which is self-explanatory, needing no further defining, should be the ultimate and final choice. Inasmuch as "elastic limit" at once suggests limit of elasticity, and hence a departure from proportionality in the stress-strain curve, it includes both expressions and indicates at once the limits of the elastic theory.

It is shown that unless one standardizes procedure in every respect apparently different results are obtained. We will do well to emphasize this point. Testing laboratories throughout the country, particularly college and research laboratories, should continually seek to impress like conditions for comparable results.

But it should be kept in mind that the testing of materials and interpretation of data is the connecting link between theoretical mechanics of materials and their later use; and that while we want a complete understanding of the factors influencing tests, perfection in the shape of the specimen and in the testing apparatus and uniformity in the procedure and interpretation, the counterpart of these in practice is almost impossible to attain.

The local weakness in a test specimen and the bent specimen are to be found in the material to be placed in the structure. No material is perfectly homogeneous, and much of our structural material is warped or straightened on the job by gagging, which of course means that some part of the material is stressed beyond the elastic limit before it commences to function as intended by the designer.

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[1] Professor of Engineering Mechanics, University of Michigan, Ann Arbor, Mich.

In 1917 the Army Ordnance Manual allowed any speed of the cross-head of the testing machine up to 6 in. per minute on testing specimens of ordnance material. In this matter, as well as in uniformity of specimens, testing equipment, procedure, and interpreting, we have made reasonable advances, and the author's paper is particularly valuable in its tabulation of the various matters that affect the accuracy of the property sought. It should contribute to the cause of uniformity and also assist the less experienced in diagnosing the cause of freak results of tests.

As to basing design values on yield point instead of elastic limit there seems to be a good argument, provided we can drop our various ways of obtaining yield point and fix upon some standard like that used by the author.

In conclusion it might be interesting to take a material with a high Poisson's ratio and see if a "proportional limit" lower than the "elastic limit" cannot be obtained due to the reduction in area of the test specimen.

THE RATE OF HYDRATION OF CEMENT CLINKER

By F. O. Anderegg[1] and D. S. Hubbell[2]

Synopsis

The lack of knowledge concerning the rate of hydration of cement clinker has hindered a more complete understanding of the complicated setting and hardening reactions that cements undergo. Carefully sized fractions have been separated by prolonged elutriation with dried air and gaged with water under a variety of conditions. After varying periods of storage, samples were removed and rapidly dried. The partially hydrated material was ground in an agate mortar until a high degree of uniformity of size was secured. The ground powder was made into a slide and the ratio of hydrated to unhydrated determined.

Storage in water at 70° F. resulting in the hydration of a layer of a typical portland-cement clinker to an average depth of about one-half micron in 24 hours, 1.7 in 7 days, 3.5 in 28 and about five microns in 90 days, in the presence of calcium sulfate. In the absence of calcium sulfate, the penetration appears to be somewhat greater. Calcium chloride in the presence of calcium sulfate does not seem to have much effect on this reaction. A temperature of 32° F. slowed down the initial reaction and one of 100° F. increased it at early periods, but no appreciable difference could be detected at 28 and 90 days at either temperature from similar hydrating cement stored at 70° F. One portland cement of high early strength was apparently hydrated more completely at 7 and 28 days than ordinary cement. The greater apparent hydration of a white portland cement is explained by the larger surface area due to greater irregularity in shape.

One of the most important fundamental properties of any hydraulic cement is the rate at which it will react with water. Knowledge of this rate is necessary for the understanding of the complicated setting and hardening processes that hydrated cements undergo. It is also of importance in connecting the fineness of grinding with the resultant strength and other properties.

Previous work on the direct determination of rate of hydration has been very meager. H. Hauenschild,[3] by observing the effect of hydration on the birefringence of portland-cement particles, has reached the conclusion that those of a dimension of eight or fewer microns were completely hydrated within a short time. It should be pointed out, however, that the interference colors of most of the minerals in portland-cement clinker are quite feeble and, with his method of observation would very soon be obscured by the products of hydration. Preliminary experiments with material of this size indicated that Hauenschild's conclusion was quite incorrect, only particles of less than two microns diameter being completely hydrated within 24 hours. The subject seemed of sufficient importance to justify a more careful determination of the rate of hydration of cement clinker.

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[2] Industrial Fellow, Atlas Portland Cement Co. Industrial Fellowship, Mellon Institute of Industrial Research, Pittsburgh, Pa.

[3] H. Hauenschild, "The Particle Size of Portland Cement Flour and Its Effect on the Rate of Reaction," *Zement*, Vol. 15, pp. 453, 469, and 488 (1926).

Most of the experiments were carried out with a standard portland cement clinker, portland cement A, of average composition. In addition, another standard portland cement, cement B, one of high early strength, cement C, and a white portland cement clinker have been included.

SEPARATION OF FRACTIONS

The separation of the cement into fractions of a sufficiently narrow range was accomplished in a Pearson elutriator[1] by prolonged blowing. At least one week of elutriation was required for each separation. The most important point in successful separation with this apparatus is to maintain the tube in very active vibration. This was secured by impacts from a small swinging hammer attached to the periphery of a disk revolving in the middle of a battery of three elutriators.

The stream of air issuing from a nozzle with an opening 2 mm. in diamater was directed downward against the cone-shaped bottom of the elutriator. The air was first scrubbed by passing through a cylinder packed with cotton waste. It was then dried by contact with 20 layers of calcium chloride supported on woven wire in a large steel drum. The velocities used were in the range where the relation between the pressure of the air and the dimension of the particle is linear and covered by the equation:

$$d = \frac{p}{3} + 10$$

in which d is the separation dimension in microns and p is the pressure differential in millimeters of mercury. The separation dimension was taken, with the exception of a few strays, as the largest dimension among the particles blown over, and as the smallest dimension among the residual particles. This was observed by measuring the images of the particles projected on to a screen with a magnification of 4400 times.

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[1] J. C. Pearson and W. C. Sligh, U. S. Bureau of Standards, *Technologic Paper No. 47* (1915).

After separation of cement clinker of the dimensions shown by preliminary experiments to be most suitable, ranging from about 15 to about 35 microns, into 8 fractions of 20 to 30 g., each fraction was subjected to further blowing, starting with the largest particles. Any cement blown over was added to the fraction of next smaller size. Two fractions of larger diameters were obtained in a similar apparatus but having a smaller diameter (1.75 in.). Figures 1 and 2 show the typical size of particles in two of the fractions.

GAGING AND STORAGE

About 2 to 3 g. of cement were mixed neat with about 30 per cent of water to normal consistency with one minute of working with a spatula. It was then packed into a small ($\frac{1}{4}$ by 1 in.) test tube and stored in a thermos bottle for the first 24 hours and thereafter in a minimum amount of water, so as not to lose much by leaching. It was observed that lime crystals tended to grow on the outside of some of the specimens, especially where no calcium sulfate had been added, but in amount too small to affect the final results. Most of the experiments were made at standard conditions, although the results of mixing and storage temperatures of 32° F. ± 5° F. and 100° F. ± 5° F. were also determined. The amount of mixing water, the time of mixing and the method of storage, were also varied. In addition, calcium chloride was added in three experiments.

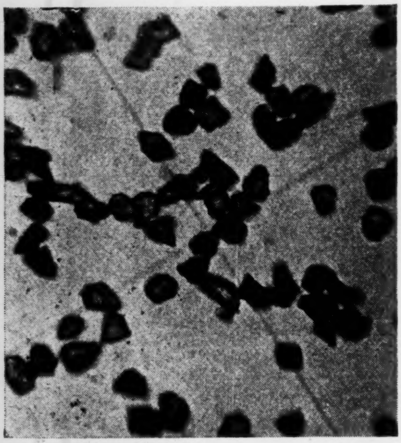

FIG. 1.—Fraction of Cement Clinker A of Mean Diameter 15.53 Microns (× 230).

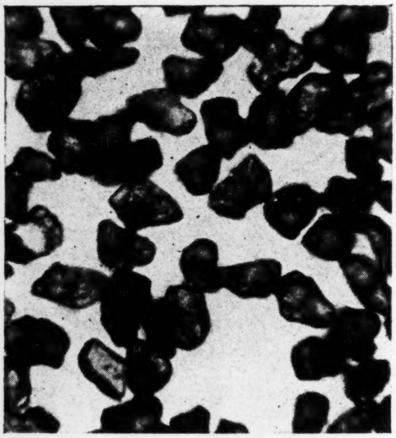

FIG. 2.—Fraction of Cement Clinker A of Mean Diameter 27.45 Microns (× 230).

Since the calcium sulfate had all been blown out of these fractions, an amount was introduced in part of the experiments, in proportion to the surface area as compared with that of a regular cement. For this purpose plaster of Paris was added, so that the SO_3 content should be increased 0.80 per cent for most of the fractions, and by a smaller amount for two fractions of larger dimension.

DRYING AND COUNTING

Samples about the size of a pea were broken from the stored specimens at definite intervals and were dried at 110° C. to stop hydration. That the hydration of the gaged fractions is completely stopped by this method was concluded as a result of the observation that a sample gave a count one month after drying that checked within 0.5 per cent., not only of the count determined immediately after drying, but also of the count made on the same material from which only the superficial water had been allowed to evaporate at room temperature. These observations also supported the conclusion that the index of the hydrated material had not been raised above 1.67 by this method of drying.

The first step in determining that portion of the clinker which had become hydrated was accomplished by grinding small bits from these dried fragments in a small, highly polished agate mortar as finely as possible, and to that point (determined by the microscope) where the fragments were very nearly the same size and very few if any remained that were appreciably larger than the others. That this was actually accomplished was determined by the actual measurement of the area of all the particles contained in one quadrant in about 20 fields, selecting every twenty-fifth field at random in each of three slides, all typical of those used for the counting.[1]

For the purpose of counting, three or more slides were made of some of this dried, partially hydrated material in an oil whose index of refraction was 1.67, a value lying between those of the unhydrated cement minerals and those of the products of hydration.[1]

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[1] The images were projected on to a ground glass plate at a magnification of 510 diameters, and areas corresponding to certain definite size ranges were determined by comparing with standard areas. There were 4998 particles in the first range between 2.5 and 5 square microns, 306 between 5 and 7.5, 10 between 7.5 and 10, none in the fourth range and two in the region lying between 13 and 16.7 square microns. The amount of material of area below 2.5 square microns was found, with the aid of the ultramicroscope, to be a negligible part of the whole. With the dark field illumination it was observed that the particles were actually composed of aggregates of very fine material clumped together during the preparation of the slides into agglomerations of quite uniform size. The individuals in any group were largely either unhydrated clinker or hydrated cement, but the separation was not complete. A check on the method of counting was then made by taking three known mixtures of unhydrated clinker and nearly completely hydrated cement (checked by a separate count). One of the mixtures contained mostly hydrated cement, one mostly unhydrated clinker and in the other the two were present in about equal portions. The calculated checked the observed in each case within 2.5 to 3 per cent, which appears to be about the accuracy of the method. This means that the error in the depth of hydration is probably ±0.2 micron.

The total number of particles and the number lying above (or below, whichever were fewer) the index of the oil were counted accurately, a total of about one thousand particles being observed in each case. In the comparatively few instances where particles were found of area somewhat greater than that of the range in which most of the material lay, an effort was made in the counting to assign the proper weight. For instance, a particle of double the area of the typical particle would be counted as 3 particles.

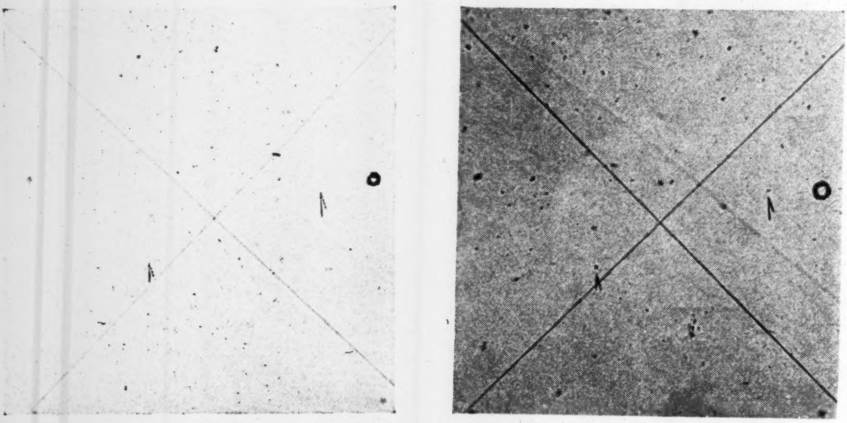
(*a*) (*b*)

FIG. 3.—Determination of Amount of Hydration (× 230).

The material is carefully ground and made into a slide with an oil of index of refraction 1.67. In Fig. 3(*a*) the particles are nearly brought up into focus, most of the particles appearing light. In Fig. 3(*b*) the tube has been elevated slightly so that the light particles have changed to dark ones These are the hydrated particles, some of which are indicated by arrows. The unhydrated particles, around some of which circles have been drawn, have turned from dark to light. These photomicrographs give a fair indication of the uniformity with which the materials are ground. Results made by such counts have been repeatedly duplicated.

The procedure of comparing the indices with that of the oil was, briefly, that known as the Becke method, or rather a modification of it. The slide was placed on the stage of the microscope (using the 4-mm. objective and the 10 occular) and the substage diaphragm was stopped down almost completely. The fragments were brought into focus as shown in Fig. 3 (*a*) and then the tube was raised slightly by the fine adjustment. Such elevation caused each particle either to

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[1] A. A. Klein and A. J. Phillips, "The Hydration of Portland Cement," *Journal*, Washington Academy of Science, Vol. 4, p. 573 (1914); also U. S. Bureau of Standards *Technologic Paper No. 43* (1914).

TABLE I.—RESULTS OF TESTS ON HYDRATION OF CEMENT CLINKER.

Mean Dimension, microns	SO_3 Added, per cent	Gaging Method	Storage Method	Percentage of Clinker Hydrated				
				24 hours	41 hours	7 days	28 days	90 days
PORTLAND CEMENT A								
15.53..........	None	Standard	Standard	....	33.7	70.0	89.1	92.4
16.53.........	None	Standard	Standard	....	13.1	66.4	81.6	88.3
	0.80	Standard	Standard	17.1	....	57.2	72.7	84.1
	None	Standard	Standard	....	7.5	65.0	....	....
18.67.........	None	Standard	....	....	21.5	63.4	79.8	85.6
	None	Standard	Dry after 7 days	....	24.0	61.3	74.2	86.8
	None	Retempered after 30 minutes	Standard	....	10.8	50.4	81.5	87.6
	0.80	Standard	Standard	....	....	43.5	69.0	90.0
20.69.........	None	Standard	Standard	9.5	....	69.6	78.9	90.3
	None	1.4 N. C.	Standard	....	19.0	68.2	84.7	89.2
	0.80	Standard	Standard	9.7	....	43.7	68.5	85.9
22.05.........	None	Standard	Standard	....	23.0	60.0	83.3	88.7
	0.80	Mixed 10 minutes with spatula	Standard	13.0	....	43.6	73.9	87.1
	None	Standard	Standard	7.3	....	28.5	67.3	85.9
	None	100° F.	100° F.	33.1	....	....	80.1	87.6
	None	32° F.	32° F.	7.6	....	....	80.7	88.3
23.57.........	None	Standard	Standard	....	18.8	61.2	76.7	86.9
	None	Standard	Standard	....	14.2	55.4	80.5	88.2
	0.80	2 per cent $CaCl_2$ solution	Standard	14.5	....	21.2	57.8	78.3
	0.80	Standard	Standard	10.3	....	33.0	68.6	86.6
25.53.........	None	Standard	Standard	....	16.8	52.3	79.5	84.1
	0.80	Standard	Standard	16.3	....	42.7	77.6	83.0
25.60.........	0.88	100° F.	100° F.	14.8	....	47.2	....	....
	0.72	32° F.	32° F.	5.7	....	18.7	....	....
38.60.........	0.27	Standard	Standard	2.0	....	19.5	....	....
	0.27	$CaCl_2$=2 per cent of cement	Standard	6.0	....	25.2	....	....
	....	....	3 hours	....	4 days	....	....	....
63.33.........	0.10	Standard	Standard 3.3	6.0	8.5	12.5	....	....
	0.10	$CaCl_2$=2 per cent of cement	Standard 4.5	7.0	8.5	12.0	....	....
PORTLAND CEMENT B								
26.73...........	0.80	Standard	Standard	5.8	....	26.0	....	....
PORTLAND CEMENT C								
27.13...........	0.80	Standard	Standard	10.0	....	47.2	78.1	....
WHITE PORTLAND CEMENT								
27.98...........	0.80	Standard	Standard	15.7	....	55.1	78.5	....

brighten or darken according as its index was above or below that of the oil, as indicated in Fig. 3 (*b*). The work of counting was greatly facilitated by employing a Euscope by means of which the image is projected on a screen where it could be viewed with both eyes. The screen was divided into ½-in. squares for ease of counting.

After a little practice very uniform results were secured in counting, which were frequently checked by two operators working independently, or by the same observer after a period of time. Typical examples of the results for the amount of unhydrated material, selected at random were: 34.1, 32.9 and 33.5 per cent for three slides made from the same material. Lack of space prevents recording these data in detail.

ESTIMATION OF PERCENTAGE OF ORIGINAL CLINKER HYDRATED

Examination of the cement after separation, but before it was mixed with water, indicated the presence of a few grains of calcite and other minerals having indices below 1.67. The percentage of these grains was determined by actually counting several hundred particles of each fraction. This correction was of the order of 2 per cent and has been included in the results given in Table I. The absence of more than mere traces of hydrated cement in the material at the start of the experiment was indicated by these observations, evidencing the effectiveness of the drying of the elutriation air supply.

This method of counting leads to an estimate of the relative volumes of the hydrated and unhydrated cement. When the clinker is hydrated expansion takes place, requiring a correction for the change in specific gravity in order to determine what percentage of the original clinker has been hydrated. With dry kerosine in a small pycnometer, the specific gravity of the original clinker was found to be 3.192. A partially hydrated cement, in which the ratio of hydrated particles to total particles was 0.853, as determined by counting, had a specific gravity of 2.347 after the usual drying. The specific gravity of the completely hydrated cement after drying at 110° C. would then be 2.13. This correction has been applied to the results given in Table I.

Incidentally, these specific gravity determinations indicate a bulking of something like 50 per cent during the hydration of portland-cement clinker.

DEPTH OF HYDRATION

Assuming that the core of unhydrated cement remaining after a given period of hydration is similar in shape to the original particle, the depth of penetration of the hydration reaction is calculated by multiplying the cube of the mean dimension by the percentage of unhydrated material and extracting the cube root. This gives the mean dimension of the core and the depth is one-half the difference between it and the original mean dimension.

The *mean dimension* was obtained by dispersing a small amount of a given fraction on a slide with absolute alcohol. The alcohol on evaporating left the particles adhering to the glass. The images of the particles were projected onto a screen at a magnification of 4400 times. The fairest long and short dimensions were measured for some 150 particles and the mean dimension was obtained therefrom by combining the average of larger dimensions with that of the smaller, the latter being taken twice. In the corner of Fig. 4 two dimensions of 150 particles of the mean dimension of 18.6 microns are plotted.

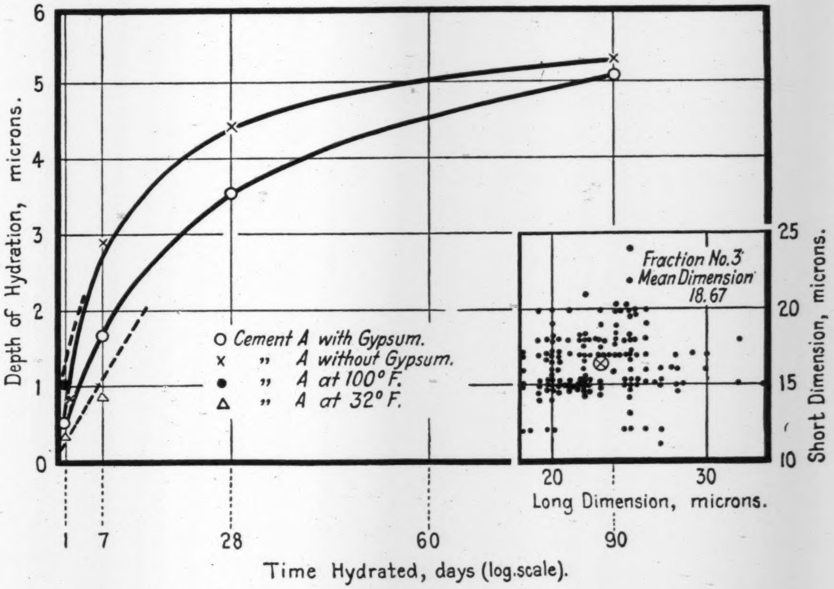

FIG. 4.—Showing Depth of Hydration of a Portland-Cement Clinker after Various Periods of Storage, With and Without the Addition of Calcium Sulfate.

In one corner the actual size of 150 particles of clinker of mean dimension 18.67 microns is shown, the smaller dimensions being plotted as ordinates and the larger as abscissa.

Attempts have been made to get a fair vertical dimension by focusing first on the surface of the slide and then on the top of each of several hundred grains. For this purpose a new and carefully calibrated micrometer screw was available. The average of several hundred such measurements has always been greater than the average of the smaller horizontal dimension. Now it seems logical to assume that the average of the vertical dimension should be the smaller, and the reason for the discrepancy lies in the fact that the highest point of each particle is brought into focus, while in the measurement of the images, the fairest dimensions are taken. Therefore, instead of averaging the horizontal long and short dimensions with the vertical dimension it seemed fairest to use the smaller horizontal dimension twice together with the longer in calculating the mean dimension. It might be stated that the number of observations by the focusing method was sufficient to overcome the errors in measuring individual grains.

Other methods of obtaining the most probable dimension, such as that of measuring a single horizontal dimension[1]; or treating the observations by the method of least squares, or by plotting probability curves, were found by actual trial to bring about only negligible improvements in the final result. To obtain some idea of the magnitude of this effect, calculations have been made for each one of 150 particles in the fraction of mean dimension 25.53 microns, where the scattering was greater than in most fractions. For depths of hydration of one and five microns the correction was found to be about 5 per cent. The error due to the eccentricity and scattering of a properly sized fraction is not very great.

With a 10-per-cent hydration the thickness of the hydrated layer for particles of mean dimension 22 microns would be 0.39, and would reduce to 0.37 micron, if the dimension had actually been 23 microns. At 90 per cent hydration the respective values would have been 6.34 and 5.85. A small error in the mean dimension will not have much effect on the final result.

CONCLUSIONS

The rate of hydration is rapid at first and then falls off, as is to be expected from the decreasing area of unhydrated clinker and from the ever thickening envelope of hydrated gel covering the surface.

The presence of calcium sulfate appears to slow up the rate of hydration between the first and twenty-eighth day of storage, as shown graphically in Fig. 4.

At 24 hours the depth reached by the hydration of a typical portland cement clinker is about 0.5 micron, at seven days about 1.5, at 28 above 3.5, and at 90 it appears to be about five microns, in the presence of calcium sulfate, and for particles of 15 to 30 microns dimension.

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[1] Henry Green, "The Effect of Non-Uniformity and Particle Shape on Average Particle Size," *Journal*, Franklin Inst., Vol. 204, p. 713 (1927).

Any variations observed due to the presence of calcium chloride are within the experimental error, therefore the preliminary conclusion might be drawn that calcium chloride does not affect the rate of hydration. This should receive more study.

TABLE II.—DEPTH OF HYDRATION.

All values are expressed in terms of microns.

Mean Dimension, microns	24 hours	41 hours	7 days	28 days	90 days
CALCIUM SULFATE ADDED					
PORTLAND CEMENT A:					
16.53	0.51	....	2.03	2.41	3.84
18.67	....	....	1.62	3.01	5.00
20.69	0.36	....	1.81	3.31	4.97
22.05	0.50	....	1.92	3.98	5.46
23.57	0.42	....	1.47	3.77	5.75
25.53	0.73	....	2.16	5.01	5.69
38.60	0.13	....	1.35	....	....
63.33	0.65	....	1.38	....	....
Average	0.47		1.71	3.54	5.12
PORTLAND CEMENT B:					
26.73	0.27	....	1.27	....	....
PORTLAND CEMENT C:					
27.13	0.43	....	2.60	5.37	....
WHITE PORTLAND CEMENT:					
27.98	0.77	....	3.27	5.61	....
NO CALCIUM SULFATE ADDED, CEMENT A					
15.53		1.00	2.56	4.06	4.48
16.53		0.39	2.52	3.57	4.23
16.53		0.28	2.44	....	....
18.67		0.72	2.67	3.86	4.44
20.69		0.35	3.41	4.17	5.59
22.05		0.92	2.85	4.95	5.70
22.05		....	1.95	3.58	5.94
23.57		0.80	3.19	4.54	5.80
23.57		0.58	2.78	4.95	6.00
25.53		0.76	2.79	5.24	5.85
27.45		1.41	4.33	5.81	7.42
Average		0.76	2.86	4.39	5.55
SPECIAL CONDITIONS					
$CaCl_2$, 2 per cent of cement and sulfate		0.39	1.78 1.32	....	....
$CaCl_2$, 0.7 per cent of cement and sulfate		....	0.90	2.94	4.70
1.4 Normal Consistency		0.71	2.64	4.82	5.42
Retempered after 30 minutes		0.40	1.95	4.01	4.68
Stored dry after 7 days		0.82	2.53	3.39	4.58
Mixed 10 minutes		0.50	1.92	3.98	5.46
Mixed and stored at 32° F		0.28 0.37	.... 0.85	4.66 4.3	5.63 ....
Mixed and stored at 100° F		1.38 0.66	.... 2.45	4.59 4.7	5.53 ....

Hydration over short periods of time has apparently been slowed up by the low temperatures, and hastened by higher temperatures;

but after storage of 28 and 90 days, the amount of hydration is apparently very nearly the same over the temperature range, 32 to 100° F.

The high early strength portland cement C did not appear to have a greater rate of hydration at 24 hours than the others, although at 7 and 28 days an appreciably greater rate was observed. The 24-hour result has been repeated.

The white portland cement appeared to react more rapidly than the gray cements. One reason for this was the much greater irregularity of the white clinker particles actually observed under the microscope.

A preliminary estimate gives the increase in volume of portland cement during hydration as about 50 per cent. As the hydration of the cement continues, the pores of a concrete should be diminished by the swelling cement. Given the size distribution of a cement, it should be possible to estimate the decrease in pore volume and in permeability with age caused by this swelling, but of course the calculation would not hold if material is transported and redeposited by moisture passing in and out.

The hypothesis of Giertz-Hedström and Werner[1] that the compressive strength of cement and concrete is proportional to the square of the part of the cement hydrated is confirmed in the case of cement A for ages of 1, 3, 7 and 28 days with specimens made according to A.S.T.M. specifications. Using the values given in Table II and knowing the size distribution, it is calculated that cement clinker A, a widely marketed brand, is 24 per cent hydrated in 24 hours, 42 per cent at 7 days, 51 per cent at 28 days, and 60 per cent at 90 days. Dividing the compressive strength in pounds per square inch by the square of the fraction of the cement hydrated gives a value constant well within the experimental error, 16,700 ± 500. It should be pointed out that this confirmation is probably accidental; for instance, on increasing the water-cement ratio the hydration would not be decreased, while the early strengths are affected more than later ones.

The work is being continued with portland cements and also with the pure minerals found in cement clinker.

Acknowledgments.—Grateful thanks are due Mildred Staples for the patient and skillful manner in which over one hundred thousand particles have been actually held under observation; also to Mr. F. W. Ashton for reading and criticizing the manuscript.

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[1] Stig Giertz-Hedström and Donovon Werner, "Strength Increase of Cement and Concrete," *Teknisk Tidskrift, Upplaga C,* Vol. 58, Kemi 6, p. 41 (1928).

DISCUSSION

MR. R. H. BOGUE[1] (*presented in written form*).—A study of this nature is attended with so many difficulties and involves so great an amount of exacting measurements that much credit is due the authors for the painstaking examinations which they have made. The subject of hydration of cement clinker is of such importance that every effort to learn more of the reactions with water is of value to the advancement of the cement industry.

A critical examination of the method used indicates that there may be a few points which should be explained or given further consideration before the described procedure can be generally accepted.

1. The authors report that the fragments of set cement were ground to essentially the same size and substantiate the statement by the experimental observation that about 94 per cent of 5316 particles were found to have an area between 2.5 and 5.0 square microns. It is implied that sizes between 2.5 and 5.0 square microns may be considered identical for the purposes of this study. This assumption may involve an error of some magnitude, as shown by the following analysis.

It is reasonable to assume that the unhydrated material in a partially hydrated cement may be of a different order of hardness or brittleness than the hydrated material. Hence, it might be expected that the mean size of the two types would differ when the two are ground together in an agate mortar.

Assume that the mean diameter of the hydrated particles actually is 2.0 microns and of unhydrated particles 2.5 microns. These values correspond approximately to areas of 3 square microns and 5 square microns, respectively, which brings them within the limits regarded by the authors as similar.

Assume that an equal number of each type are present and counted. The volume of the grains, assuming an average spherical shape, is given by the formula:

$$V = \frac{4}{3}\pi r^3 = 4.19\ r^3$$

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[1] Research Director, Portland Cement Assn. Fellowship at the U. S. Bureau of Standards, Washington, D. C.

Hence, the mean volume of the hydrated grains is 4.19 cubic microns and that of the unhydrated grains 8.16 cubic microns. The weight is equal to the volume times the density. Hence, the weight of the hydrated grains is $4.19 \times 2.13 = 8.19^{-6}$g. and the weight of the unhydrated grains is $8.16 \times 3.192 = 26.1^{-6}$g. Then the percentage by weight of hydrated material in the sample is $8.91/35.01 = 25.4$ and of the unhydrated material, 74.6. If the volumes were considered to be identical, as is assumed by the authors, the weight percentages would be proportional to the densities, or $2.13/5.322 = 40$ per cent hydrated and 60 per cent unhydrated. Thus, a possible difference of 14.6 per cent in the estimation of the material may result from this cause alone, and within the size limits which the authors regard as similar. The 6 per cent of particles falling outside of that size range would result in further error.

2. It was found that the particles actually are aggregations of very fine material. This being the case, it seems improbable that the individual grains comprising the aggregation will be present in all particles in equal numbers and sizes. If the number and size of the grains vary in the different particles, then the mass of the particles must vary proportionately, even assuming equal external dimensions, equal hydration and identical compounds, whereas it has been assumed in the paper that the particles of similar size will have the same mass if all are hydrated (or unhydrated).

3. The paper points out that the individuals in any group of particles consisted largely of either hydrated or unhydrated material. No attempt was made to measure or to estimate the portion of the two types of material in each grain, but the whole particle was rated either as hydrated or as unhydrated. Therefore, the statement that the particles consisted largely of one type or the other seems open to question as an argument that no significant error is introduced by ignoring the observed inhomogeneity.

4. A mean density for hydrated (or unhydrated) cement is established for use in the calculations. Portland cement is not a homogeneous material. The grains of powdered clinker consist of several compounds. These are known to react with water at vastly different rates. Thus, the hydrated material at one or two days may be largely tricalcium aluminum hydrate with a small amount of hydrated tricalcium silicate. It is improbable that the density of this hydrated material is similar to that of a completely hydrated cement, as is assumed in the paper. Likewise, the unhydrated material, after a month, may consist largely of dicalcium silicate which has a density that differs somewhat from the density of clinker.

5. The term "depth of hydration" as used in the paper conveys a picture of the hydration process that is misleading as it implies that the ground particles of cement clinker remain essentially discreet when subjected to the action of water in the consistency of mortar. Otherwise, the phrase "the thickness of the hydrated layer for particles of mean dimension . . ." could not be used. The phrase also implies that the water encroaches upon the grain in such a manner as to result in the formation of a more or less uniform layer of hydrated material surrounding a core of unhydrated material. But, as previously pointed out, the particles of ground clinker are not homogeneous, but consist of grains of different compounds which react with water at quite different rates. Some of these grains, in a given time period, will be completely hydrated, others will be scarcely touched, and still others will be penetrated by the selective hydration of the more readily reacting compounds. For this reason, and also because the hydration results in an increased volume of solid phase, the clinker grains disintegrate in the presence of water and lose their identity. And, of the remaining grains of individual compounds, some will be completely hydrated during a time period in which others are not perceptibly etched. Hence, the term "depth of hydration" leaves an impression of the hydration phenomenon that seems to be quite at variance with known behavior.

The authors believe that many of the errors introduced, as pointed out in this discussion, are eliminated statistically because of the large numbers of individual particles observed. They find by three counts made on known mixtures of previously determined partially hydrated cements, that the calculated values checked the observed values to 2.5 to 3.0 per cent, corresponding to a "depth of hydration" of about 0.2 micron and report this as the probable accuracy of the method. Perhaps a more appropriate basis for noting the conformity of the method (there is no means for determining the absolute accuracy of the method) is a comparison of the values obtained for the "depth of hydration" of the several sizes at some given time period. The paper justifies such a comparison by averaging these values (see Table II) and giving in the conclusions certain "depths of hydration" of typical cement clinker "for particles of 15 to 30 microns dimensions." The variation at 24 hours (calcium sulfate added) is from 0.13 to 0.73 micron or about 460 per cent. At 90 days the variation is from 3.84 to 5.75 microns, about 50 per cent. Likewise, the variation at 41 hours (no calcium sulfate added) is from 0.28 to 1.41 microns, about 400 per cent and, at 90 days, from 4.23 to 7.42 microns, about 75 per cent. The other columns of Table

II show variations intermediate between these values. It would seem from these figures that the conformity of the data obtained by the method employed was of the order of a 50 to 400 per cent variation. By applying the principle of least squares to these later data, it is found that the "probable error" at 41 hours is ± 0.23 micron or 32 per cent and, at 90 days, ± 0.64 micron or 12 per cent. These values, however, indicate only the probability of conformity in a number of determinations and do not in any manner express the probable accuracy of the method.

The writer feels that the method suggested may have an important field of usefulness as an independent means for hydrate determination, provided the difficulties above discussed can be evaluated in the calculations. He feels, however, that until the procedure has been more fully developed, it would be premature to place complete confidence in the data obtained.

MR. J. C. WITT[1] (*by letter*).—This paper presents an advance in a field which is becoming more and more important to the cement manufacturer and user. The degree of division of cement must receive more attention in the future. The expression of this property in terms of fineness is only arbitrary, the important consideration being the surface per unit of mass. It is likely that eventually fineness will be replaced by "square feet per barrel," or some equivalent term.

In the opinion of some investigators there is a critical fineness for cement—that is, a point beyond which no advantage is to be gained. This idea remains to be demonstrated, but the determining factor is obviously the velocity of hydration, or the degree of division which will permit hydration to reach the desired value in a given time and under given conditions. The paper indicates that for some time to come it is unlikely that commercial cements will exceed this fineness.

In a pioneering paper such as this, dealing with quantities with which direct measurements are difficult (and sometimes impossible) assumptions can not be avoided. Some of these assumptions will no doubt be verified later, while others may be shown to be incorrect. The ingenuity of the procedure as well as the painstaking measurements are commendable. Such information as the authors have started to obtain is greatly needed in the calculation of concrete mixes. The present practice has developed largely from cut and try methods, rather than from the application of scientific principles.

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[1] Chemical Engineer, Chicago, Ill.

MESSRS. F. O. ANDEREGG[1] and D. S. HUBBELL[2] (*authors' closure by letter*).—Referring to Mr. Bogue's comments, his consideration of points 1 to 4 has really been anticipated by us through repeated checks and counterchecks, as described in our paper. The accuracy of our method has been established by direct experiments, which answer completely the criticisms of the method of analysis reported, for if several known mixtures of various proportions of hydrated and unhydrated material can be checked to within 2.5 per cent, possibilities of error, as suggested by Mr. Bogue, have small basis of actual fact.

In his discussion of "depth of hydration," Mr. Bogue has failed to appreciate the significance of this conception to the science of portland cement. The statistical mean depth of penetration *is* the important factor in the explanation of the behavior of any commercial cement in relation to its development of strength, water-tightness and other properties; and the estimation of the amount of clinker hydrated within a given time is a very necessary detail in unraveling the complicated setting and hardening processes. In our opinion the rate of hydration as determined by our method is far more important than the true rate of hydration of a mineral intimately in contact with water. In cementing practice this condition exists for a very short while only, after the mineral surfaces have been exposed to water. The important thing to know is what the rate of hydration is when the core is protected with the ever thickening coat of hydrated material.

We do not believe there is weighty testimony to show that "clinker grains disintegrate in the presence of water and lose their identity," as Mr. Bogue contends. Mr. Bogue points out a variation between 0.13 and 0.73 micron, or about 460 per cent in one set of our results. The accuracy obtained is sufficient for the purpose at the present stage of development and compares with the determination of free lime and loss on ignition, or with other estimations of chemical and physical properties in so complex a system as portland cement.

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A COMPRESSION TEST FOR PORTLAND CEMENT

By O. L. Moore[1]

Synopsis

It is generally known that standard Ottawa sand mortar of plastic consistency gives test results considerably lower in compression than workable concrete of the same water-cement ratio. Otherwise standard sand is ideal for standardization purposes in that it is practically constant in grading, size and structure, does not absorb water and is established as the standard testing sand. This paper offers a practical method, developed in the Central Laboratory of the Universal Portland Cement Co., whereby the compressive strength of standard sand mortar of plastic consistency is a satisfactory measure of the strength property of portland cement in concrete.

Data are given to show that the compressive strength of plastic standard sand mortar is a function of its voids-cement ratio; that if the cement factor is expressed in the same terms in voids-cement ratio of standard sand mortar and in water-cement ratio of concrete, then the strengths are approximately equal when the two ratios are equal; that by means of a straight-line chart or equation between voids-cement ratio and water-cement ratio of standard sand mortar, the strength of the latter becomes a direct measure of concrete strength on a water-cement ratio basis. An example illustrating the method of procedure and data showing results obtained with fifty different brands of portland cement are given. A method for testing sands and a basis for predetermining compressive strength are suggested.

The method of test offers a good criterion for portland-cement strength in concrete. It utilizes a consistency which is subject to minimum error in mixing and molding, and it has interesting possibilities for standardization because of the uniform properties of standard sand.

Introduction

When the Central Laboratory of the Universal Portland Cement Co. was established four years ago it was desired, among other things, to measure the compressive strength of Universal Portland Cement in concrete. In order to test as many samples as possible from several mills, 3 by 6-in. cylinders and 0 to $\frac{3}{4}$-in. aggregate were decided upon and a method based on a fixed water-cement ratio $\left(\frac{W}{C} = 1.00\right)$ was adopted as a standard. This method has proved satisfactory so far as measuring strength is concerned, but the number of samples

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[1] Engineer of Tests, Universal Portland Cement Co., Chicago, Ill.

which may be tested, even with this size of specimen, is too limited for the purpose. Investigations were continued in the hope that a satisfactory method could be worked out whereby standard Ottawa sand might be used because of its several advantages over other sands for testing purposes: namely, it is uniform in grading, size and structure, does not absorb water and is established as the standard testing sand. The general interest in the subject also was considered a very good reason for continuing the investigations. After several unsuccessful attempts, a practical method finally was worked out which is giving satisfactory results and which it is thought may be of interest in the development of standard methods of test for concrete materials. The basic features of this method under standard laboratory conditions are described and discussed in the following pages.

DESCRIPTION OF TESTS

In the original series of tests which were repeated on five different days, four water-cement ratios (5, 6, 7 and 8 gal. of water per sack of cement) and one sample of cement were used with each of the materials at hand, as follows:

Concrete, 0 to $1\frac{1}{2}$ in. torpedo sand and pebbles, fineness modulus	5.60
Concrete, 0 to $\frac{3}{4}$ in. torpedo sand and pebbles, fineness modulus	4.85
Mortar, 0 to No. 4 torpedo sand only, fineness modulus	3.22
Mortar, 0 to No. 100 Lake Michigan beach sand, fineness modulus	0.06
Mortar, No. 20 to No. 30 standard Ottawa sand, fineness modulus	2.98

After results were available a 4-gal.-per-sack ratio was added in the case of standard sand. In outlining the test each water-cement ratio was specified and the operator was instructed to use enough sand or aggregate to give what we call a "roddable" consistency, which was defined as "not too dry to rod perfectly into the molds nor so wet that the mixing water will separate during mixing and molding." Cylinders used throughout were 3 by 6 in., but 4 by 8-in. and 6 by 12-in. cylinders also were used for one water-cement ratio in the 0 to $1\frac{1}{2}$ in. concrete. Weights per cubic foot and per liter of freshly mixed concrete and mortar were determined in $\frac{1}{5}$ and $\frac{1}{10}$-cu.-ft. measures, respectively, and used in calculating voids-cement ratios. These measures are watertight whereas some water loss from the specimen molds was observed, especially from the sand mortars, but no effort was made to correct for errors introduced in this manner.

In calculating voids-cement ratio a formula based on the batch rather than the unit of mortar or concrete was set up so that labora-

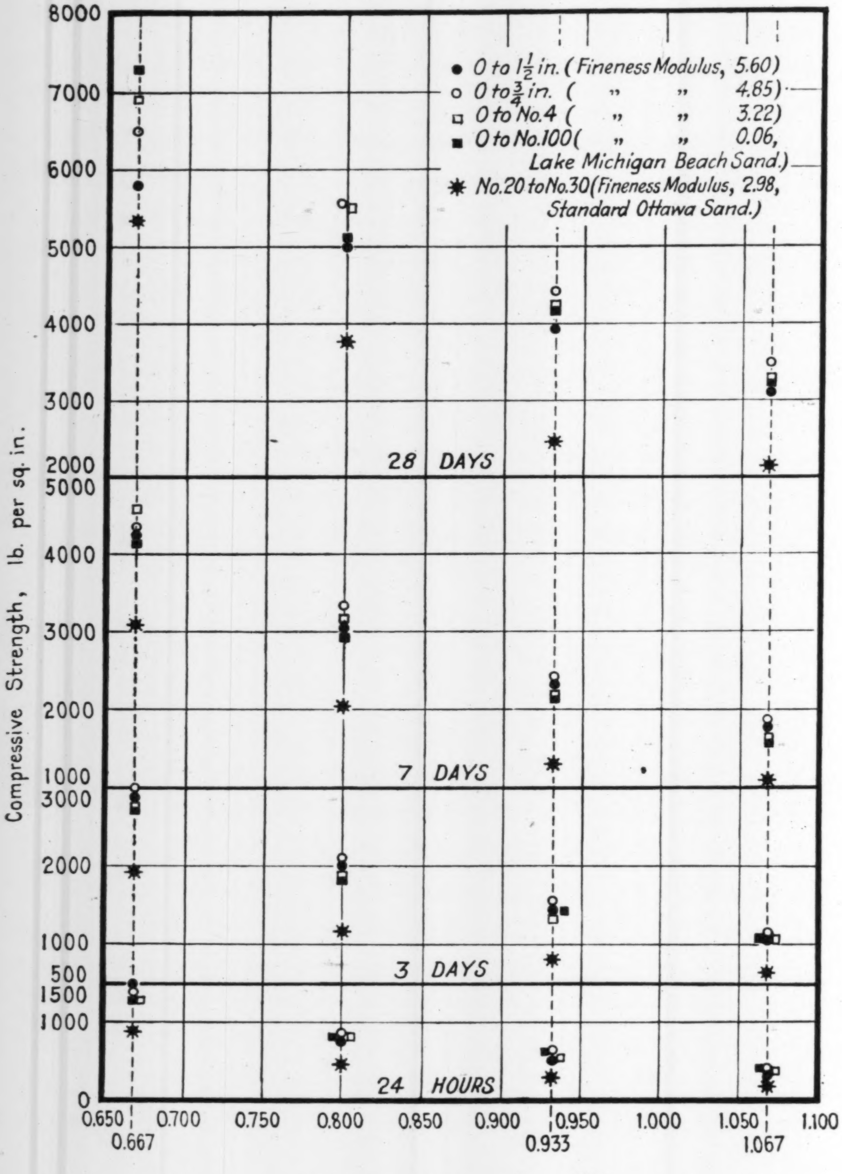

FIG. 1.—Relation of Water-Cement Ratio and Compressive Strength of Concrete and Mortar.

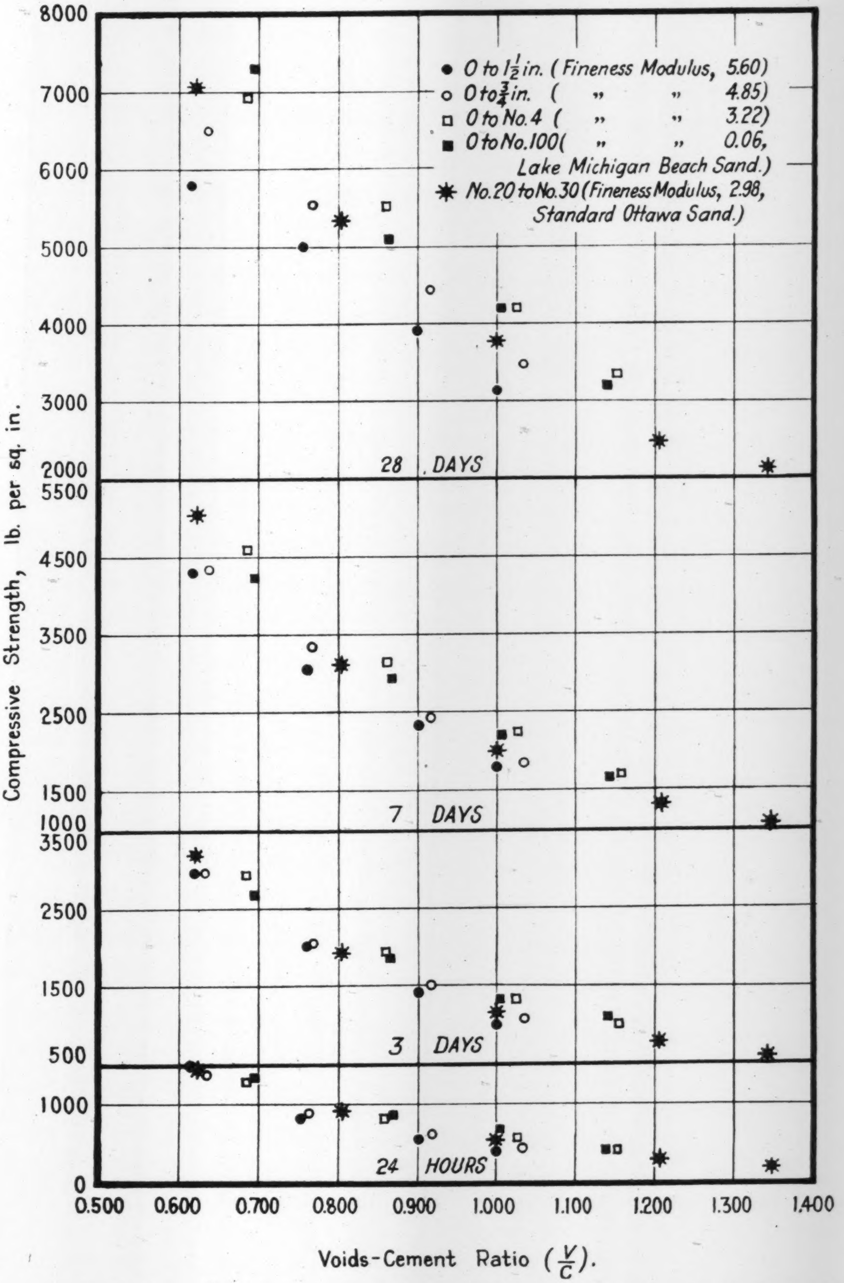

FIG. 2.—Relation of Voids-Cement Ratio and Compressive Strength of Concrete and Mortar.

tory figures on actual batches could be used directly in the formula, derivated as follows:

Let V = voids (air and water) in a batch of mortar or concrete,
C = volume of cement in a batch (based on 1505 g. = 1 liter or 94 lb. = 1 cu. ft.),
E = weight of cement in a batch of mortar or concrete,
A = weight of room-dry aggregate in a batch of mortar or concrete,
H = weight of water in a batch of mortar or concrete,
B = weight of batch = $E + A + H$,
D = weight of unit volume of fresh mortar or concrete, and
$\frac{V}{C}$ = voids-cement ratio,

then

$$V = \frac{B}{D} - \frac{A}{\text{specific gravity of } A \times \text{unit weight of water}} - \frac{E}{\text{specific gravity of } E \times \text{unit weight of water}}$$

and

$$C = \frac{E}{\text{unit weight of } E}$$

$$\frac{V}{C} = \frac{\frac{B}{D} - \frac{A}{\text{specific gravity of } A \times \text{unit weight of water}} - \frac{E}{\text{specific gravity of } E \times \text{unit weight of water}}}{\frac{E}{\text{unit weight of } E}}$$

$$\frac{V}{C} = \left(\frac{B}{D} - \frac{A}{\text{specific gravity of } A \times 1000} - \frac{E}{3.10 \times 1000}\right)\frac{1505}{E} \text{ (grams and liters)}$$

$$\frac{V}{C} = \left(\frac{B}{D} - \frac{A}{\text{specific gravity of } A \times 62.5} - \frac{E}{3.10 \times 62.5}\right)\frac{94}{E} \text{ (pounds and cubic feet)}$$

For standard Ottawa sand (specific gravity = 2.65),

$$\frac{V}{C} = \left(\frac{B}{D} - \frac{A}{2650} - \frac{E}{3100}\right)\frac{1505}{E} \text{ (grams and liters)}$$

DISCUSSION OF RESULTS

The compressive strength values are charted against water-cement ratio $\left(\frac{W}{C}\right)$ in Fig. 1 and against voids-cement ratio $\left(\frac{V}{C}\right)$ in Fig. 2, the cement factor C in the $\frac{V}{C}$ ratio being expressed as in $\frac{W}{C}$ (94 lb. = 1 cu. ft.) rather than as absolute volume. The usual low results of standard sand mortar on a $\frac{W}{C}$ basis were obtained, which is

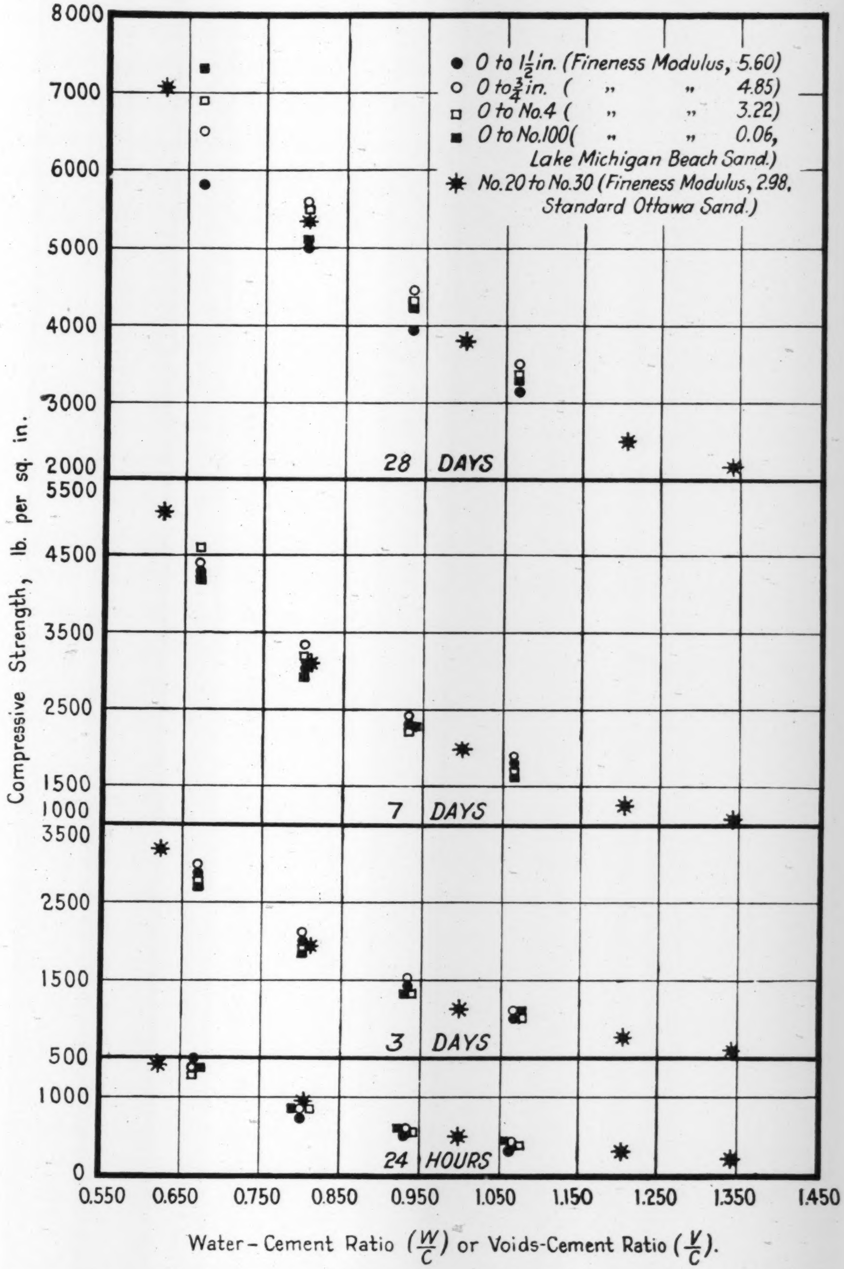

FIG. 3.—Relation of Compressive Strength and Water-Cement Ratio of Concretes, Torpedo Sand and Beach Sand Mortars, and Relation of Compressive Strength and Voids-Cement Ratio of Standard Ottawa Sand Mortars.

due to its relatively high air voids-cement ratio. On a $\frac{V}{C}$ basis the standard sand mortar falls to the right of its position on the $\frac{W}{C}$ chart and in line with the other data, leaving a vacant space at the left of the chart which was filled in later by a 4-gal.-per-sack ratio. The chart indicates that the strength of standard sand mortar is satisfactory for a strength criterion on this basis.

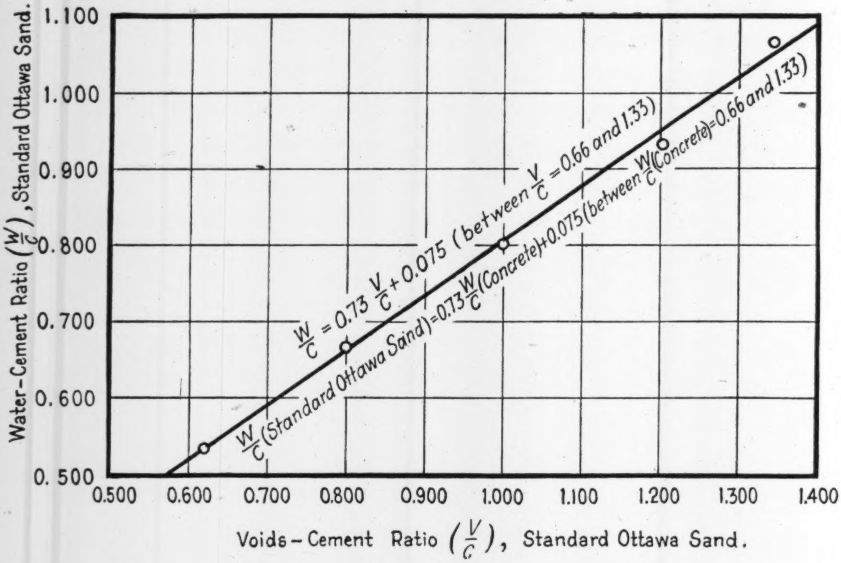

FIG. 4.—Relation of Water-Cement Ratio and Voids-Cement Ratio of Standard Ottawa Sand Mortar, Roddable Consistency.

Voids-cement ratio of standard Ottawa sand mortar may be used as the water-cement ratio of concrete for equivalent compressive strengths.

In Fig. 3 the compressive strengths of the two concretes and the two graded sand mortars are plotted against $\frac{W}{C}$ and that of the standard sand against $\frac{V}{C}$. The agreement is better than before and suggested using $\frac{W}{C}$ for concrete and $\frac{V}{C}$ for standard sand mortar as a measure of strength; but, since $\frac{V}{C}$ must be determined by trial batches and calculation, the method is not so practical for the purpose as a $\frac{W}{C}$ method would be. This disadvantage was overcome by means of an equation expressing the relation between $\frac{W}{C}$ and $\frac{V}{C}$ for "roddable" mixes of standard sand. In Fig. 4 the standard sand

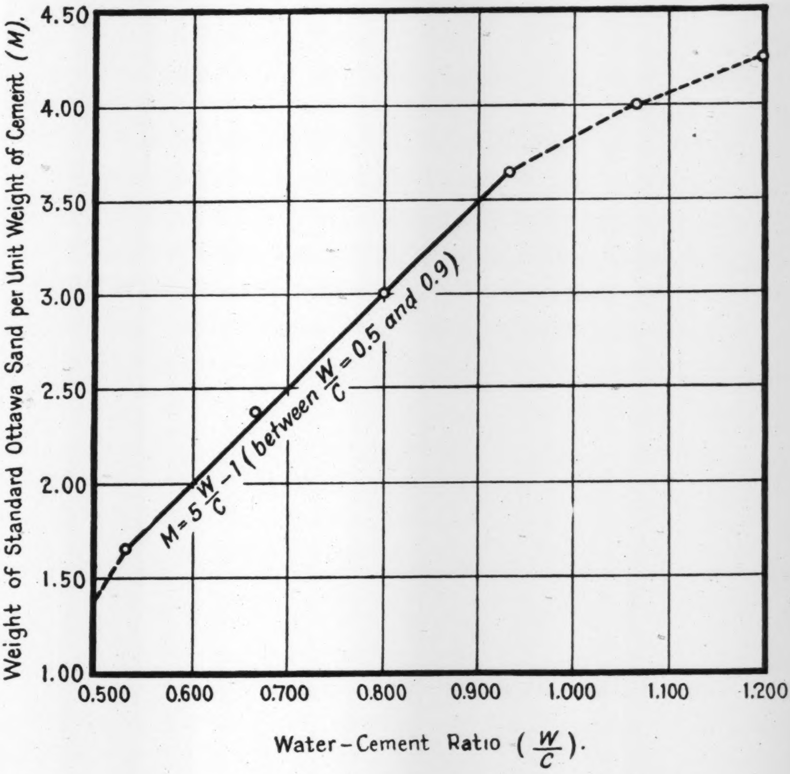

FIG. 5.—Roddable Mixes, Standard Ottawa Sand and Portland Cement.

data only are plotted, $\frac{W}{C}$ against $\frac{V}{C}$, and, assuming a straight line relation, the equation is:

$$\frac{W}{C} = 0.73\frac{V}{C} + 0.075 \text{ (between } \frac{V}{C} = 0.66 \text{ and } 1.33) \qquad (1)$$

Since it is generally accepted that the compressive strength of workable concrete is measured by either its $\frac{W}{C}$ or its $\frac{V}{C}$, and since C is expressed in the same terms in both ratios, it was concluded that the two ratios are approximately equal in normal workable concrete; and because of the agreement in Figs. 2 and 3, it was concluded that both the $\frac{V}{C}$ and the $\frac{W}{C}$ of workable concrete are approximately equal to the $\frac{V}{C}$ of "roddable" standard sand mortar when the strengths are equal. This reasoning applies to the net values as they exist in the test specimens several hours after molding, but the nominal values are close enough for practical purposes so long as water loss from molds and absorption by the aggregate are small. By substituting $\frac{W}{C}$ (concrete) in the above equation for $\frac{V}{C}$, the following equation was obtained:

$$\frac{W}{C}\text{ (standard Ottawa sand)} = 0.73\,\frac{W}{C}\text{ (concrete)} + 0.075 \text{ (between } \frac{W}{C} \text{ (concrete) equal to 0.66 and 1.33)} \qquad (2)$$

To determine the proportions of cement and standard sand to use with a given $\frac{W}{C}$ for a "roddable" mix, the following equation was set up from the data (Fig. 5):

$$M = 5\,\frac{W}{C} - 1 \text{ (between } \frac{W}{C} \text{ equal to 0.5 and 0.9)} \qquad (3)$$

where M = weight of standard sand per unit weight of cement.

The following example illustrates the method of procedure:

To measure the compressive strength of a given sample of cement in workable concrete of 7½ gal. of water per sack of cement.

7½ gal. of water per sack of cement = $\frac{W}{C}$ of 1.00; substituting in Eq. 2, $\frac{W}{C}$ (standard Ottawa sand) $= 0.73 \times 1.00 + 0.075 = 0.80$. Therefore, the $\frac{W}{C}$ to use with standard sand is 0.80.

To set up the batch, assume 500 g. of cement to be used. Then, mixing water (cubic centimeters) = cement (grams) × 0.664 $\frac{W}{C}$ (from definition of $\frac{W}{C}$) $= 500 \times 0.664 \times 0.80 = 266$ cc.

Now, using Eq. 3 (Fig. 5),

$$M = 5 \times 0.80 - 1 = 3.0$$

Therefore, the mix by weight is 1:3.0 and the batch consists of:

500 g. cement,
1500 g. standard sand,
266 cc. water.

The mortar is mixed with the hands for one minute and is rodded or spaded into 2-in. cube molds. The cubes are tested perpendicular to the direction of rodding.

Figure 6 shows compressive strengths of samples of various brands of portland cement in concrete and in standard sand mortar, using the method under discussion. Figure 7 shows results obtained in testing some of the same samples in 6-gal.-per-sack concrete. The uniformity and the agreement of the method with concrete is good and indicates a good measure of cement strength in concrete.

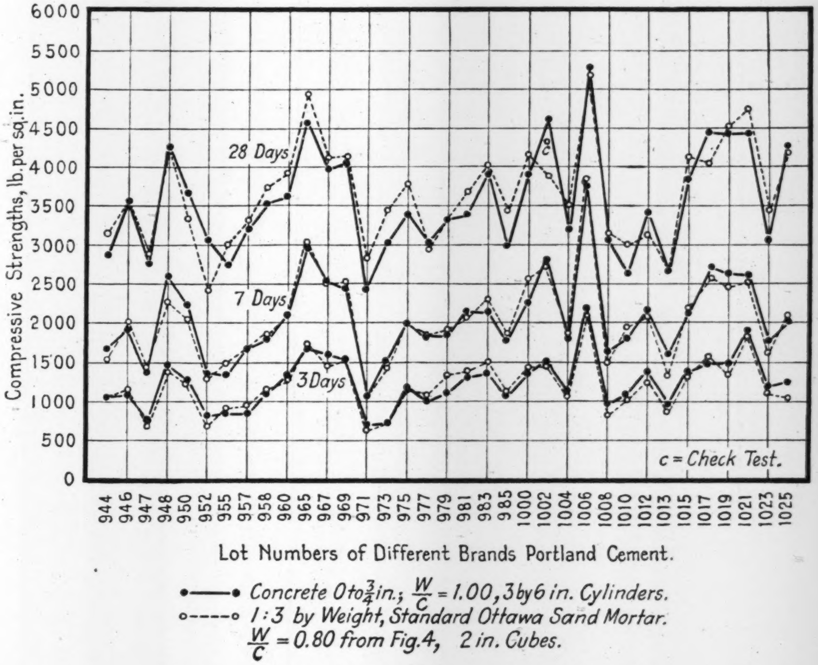

FIG. 6.—Compressive Strength of Various Samples of Portland Cement in Concrete, and of Standard Sand Mortar, Using Proposed Method.

Each point is the average of four specimens.

The effect of size and shape of test specimen,[1] the absorption of aggregates and water loss from molds should not be overlooked. The basis for strength comparison of mortar with concrete is sound, but in actual testing, differences in water loss and absorption, as well as several other factors, change the intended basis of comparison of materials.

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[1] H. F. Gonnerman, "Effect of Size and Shape of Test Specimen on Compressive Strength of Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part II, p. 237 (1925).

In our tests with surface moisture evaporated from the sand and gravel by room drying and under normal laboratory conditions, the strengths of 2-in. cubes of standard sand mortar and 6 by 12-in. cylinders of concrete are in agreement when the method is applied to concrete of 6 to 9 gal. of water per sack of cement. An occasional high result is obtained with the cube in the case of high strength in the 6-gal. ratio, as noted in Fig. 7, but no exceptions have been found in the $7\frac{1}{2}$-gal. ratio which is used as our standard.

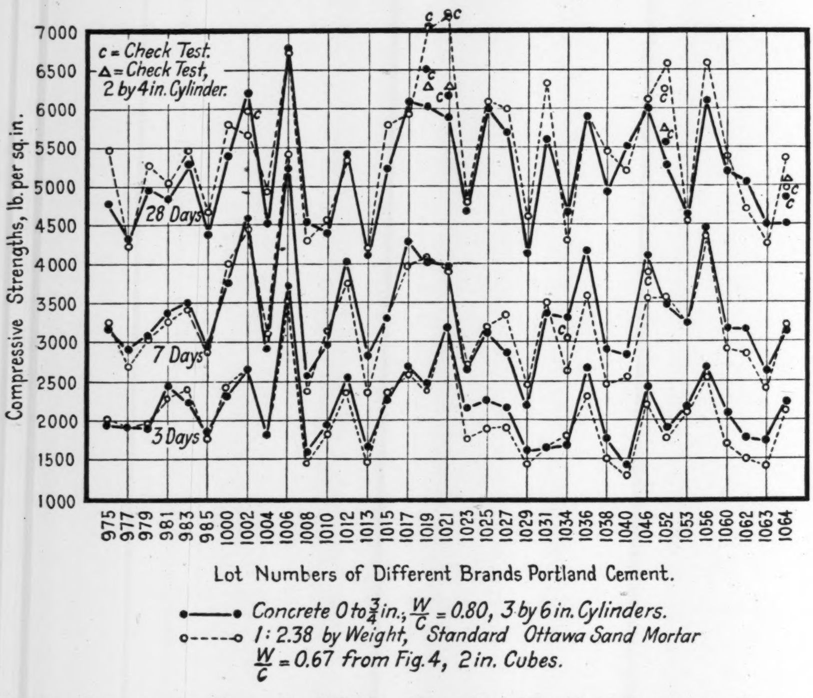

FIG. 7.—Compressive Strength of Various Samples of Portland Cement in Concrete, and of Standard Sand Mortar, Using Proposed Method.

Each point is the average of four specimens.

METHOD FOR TESTING SANDS SUGGESTED FOR INVESTIGATION

In the case of sand testing, a batch of mortar is mixed of the sand to be tested using a $\frac{W}{C}$ of 1.00 and enough sand to give a slump of 1 or 2 in. After making a weight per liter determination, the mortar is molded in 2-in. cubes. After calculating $\frac{V}{C}$ for the sand under test this is substituted in Eq. 1, $\frac{W}{C}$ calculated and parallel tests made with standard sand. The sand should give the same strength as standard sand, making allowance for test variation. The method has not been tried out thoroughly and is mentioned only as an interesting field for investigation.

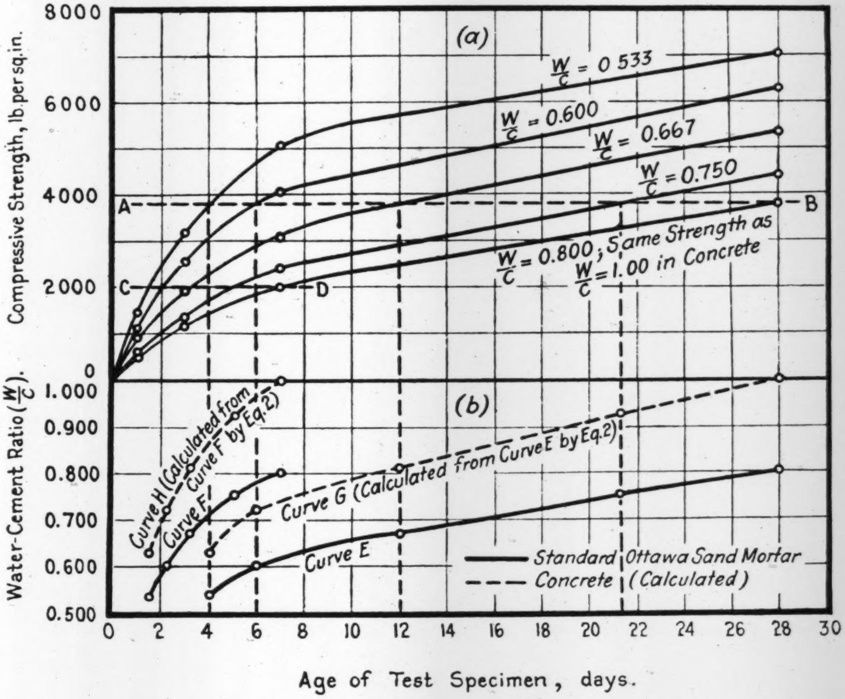

FIG. 8.—Method for Predetermining Compressive Strength.
(*a*) Compressive Strength - Age Test Specimens (Standard Ottawa Sand).
(*b*) Water-Cement Ratio - Age Curves for Predetermining Compressive Strength of Portland Cement in 7½-gal. Concrete ($\frac{W}{C}$ = 1.00).

Curves G and E, are $\frac{W}{C}$ -age combinations for predetermining 28-day strength in 7½-gal. concrete.

Curves F and H, are $\frac{W}{C}$ -age combinations for predetermining 7-day strength in 7½-gal. concrete.

PREDETERMINING COMPRESSIVE STRENGTH

As an outgrowth of these investigations, 7-day strength is being predetermined in 2 days and 28-day strength in 7 days with satisfactory results. Since concordant results are obtained with the various portland cements used in this investigation, the scheme is presented in the belief that it will be found of value to others and that it merits broader investigation.

In Fig. 8 the compressive strength values of the standard sand mortar are charted against age at test. The line *AB* intersects the several curves at points of equal strength. From these points, Curve *E* was constructed which is a series of $\frac{W}{C}$-age combinations which produce strengths equivalent to $\frac{W}{C} = 0.800$ at 28 days. $\left(\frac{W}{C} =\right.$ 1.00 concrete). In like manner Curve *F* was constructed (line *CD*) which combinations produce strengths equivalent to $\frac{W}{C} = 0.800$ at 7 days. The Curves *H* and *G* are similar relations for tests in concrete and were constructed by substituting values from *F* and *E* respectively in Eq. 2 and calculating the equivalent water-cement ratios for concrete.

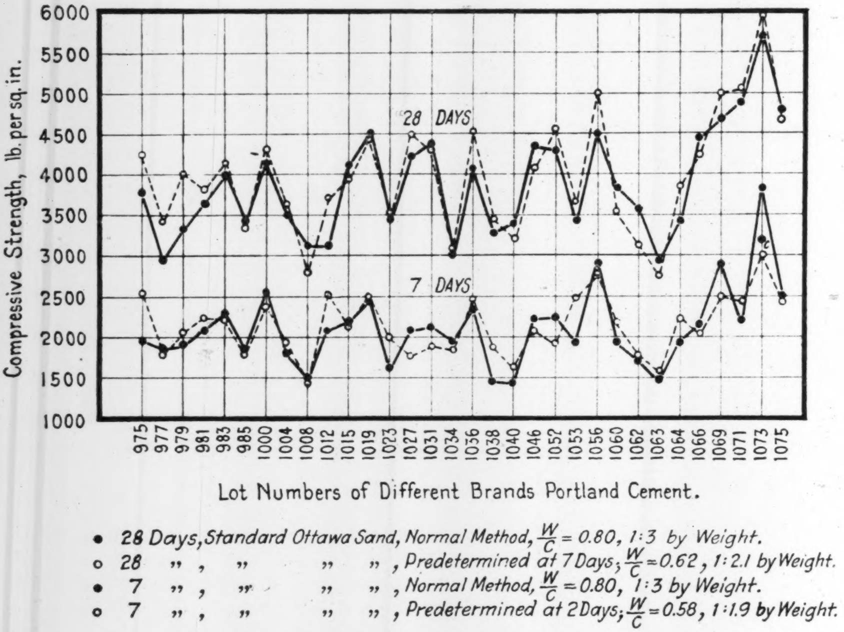

FIG. 9.—Compressive Strength of Different Brands of Portland Cement by Normal and by Predetermining Methods (2-in. Cubes).

These $\frac{W}{C}$-age relations are independent of the degree of strength of cement and furnish a basis for predetermining strength at earlier periods. Theoretically, differences in rate of early strength gain to later gain affect the predetermined values, but the effect is negligible in predetermining 7-day strength at 2 days. It is also negligible in predetermining 28-day strength at 7 days except in extreme cases. Since this effect becomes less as the predetermining period is moved toward the normal period, adjustment may be made in this manner to include cements of unusual hardening properties.

This principle as applied to samples of different brands of portland cement is illustrated in Figs. 8 and 9, where 7-day compressive strengths of standard sand mortar are predetermined at 2 days and 28-day strengths at 7 days. Tests in concrete are not included in this paper but results obtained, based on Curves *G* and *H*, are in accord.

DISCUSSION OF METHOD AS A POSSIBLE STANDARD FOR PORTLAND CEMENT TESTING

Considering Fig. 4, the relation between $\frac{W}{C}$ and $\frac{V}{C}$ for standard sand mortar is not a fixed relation but varies in the case of "roddable" mixes within rather narrow limits. For instance, with a given water-cement ratio enough sand is added to obtain a "roddable" mix. Within this rather narrow range, say, for example, from a 1-in. to a 4-in. slump, the amount of sand may be varied which varies the $\frac{V}{C}$ ratio a small amount but of course does not vary the $\frac{W}{C}$ ratio. This range does not appreciably affect the strength so that the variation is of minor consequence. Another factor which varies the $\frac{V}{C}$ ratio is differences in normal consistency of cements. This variation also appears to be small in the case of portland cements, for in these tests using fixed proportions of standard sand and cement, the various brands of portland cement produced mortars which were entirely satisfactory for molding in every case.

A relation such as Fig. 4 could be established by investigation under the direction of the Society's Committee C-1 on Cement. Such investigation could establish the range of variation in the $\frac{W}{C}$ - $\frac{V}{C}$ relation of standard sand mortar and locate the line, probably in an average position. A $\frac{W}{C}$ ratio could be chosen as standard which would correspond to a $\frac{W}{C}$ ratio in concrete, and, because of the constant properties of standard sand, the corresponding proportions of cement and sand could be fixed. Under these conditions, the consistency of the mortar varies a little with different samples of cement, but the compressive strength is measured on the same basis and errors of measuring consistency are eliminated. If, on the other hand, the consistency should be fixed, as is customary, and the $\frac{W}{C}$ ratio allowed to vary slightly, then the compressive strength varies not only with the $\frac{W}{C}$ ratio but also with errors in measuring consistency. The latter method is less desirable from a testing standpoint because it assumes that any differences in consistency of the test mortar are the same in concrete, which probably is not true, and because our present-day instruments for measuring consistency are subject to considerable error.

CONCLUSIONS

The method of test offers a good criterion for portland cement strength in concrete, at the usual and earlier periods, being based upon recognized functions of the strength of concrete. It utilizes a consistency which is subject to minimum error in mixing and molding, and it has possibilities as a basis upon which a standard compression test for concrete materials may be developed because of the uniform properties of standard sand.

DISCUSSION

MR. E. M. BRICKETT[1] (*presented in written form*).—Mr. Moore has presented some very interesting data in his paper. He has shown that in his own laboratory he can duplicate the absolute strength of concrete cylinders by means of a mortar cube test using standard Ottawa sand and proportioning the ingredients so that the voids-cement ratio of the mortar is equivalent to the water-cement ratio of the concrete.

We have studied Mr. Moore's paper at the Central Laboratory of the Lehigh Portland Cement Co. and there are several points which have come up which seem worthy of mention at this time. We have made up a number of standard Ottawa sand mortar mixes just as prescribed by Mr. Moore. It is our impression that the consistency of the mortar is so wet that segregation would result. In a paper presented last year on "A Plastic Mortar Compression Test for Cement,"[2] the mortars described were made with a graded sand. The average flow was 56 per cent on the 10-in. flow table. Later study showed that there was some segregation in these mortars. The mix was revised to a drier consistency (a flow of 140) with an improvement in results. Some of the mortars made by Mr. Moore's method gave flows above 170.

These mixes appeared harsh. This harshness is certainly attributable in part to the standard Ottawa sand with particles of nearly uniform size and shape and with comparatively high voids.

These properties characterize an undesirable mortar aggregate; the condition could be remedied by the use of a sand with at least some range in gradation. A graded Ottawa sand from the same source from which we obtain standard sand looks now to be a logical choice from the standpoint of ease of duplication and constancy of supply.

The range in consistency, as measured by flow, within which the mortar is dry enough to resist segregation and yet not so dry as to introduce an element of variation in placing is small. Mr. Moore in Fig. 5 of the paper has presented a single equation for the determination of proportions of cement, sand and water to be used for roddable mixes. Mortars made according to this figure, using several cements and several different sets of proportions, ranged in flow from 132 to 172. This is too great a range; it is due, in part at least, to the attempt to use standardized proportions with cements varying widely in plasticity and flowability. Experience has indicated to us that consistency has an important bearing on the resultant strength.

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[1] Engineer of Tests, Research Division, Lehigh Portland Cement Co., Allentown, Pa.

[2] E. M. Brickett "A Plastic Mortar Compression Test for Cement," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 432 (1928).

Mr. Moore states that the effect of size and shape of test specimen must not be overlooked. He compares mortar strengths derived from 2-in. cubes with concrete strengths derived from 3 by 6-in. cylinders and he shows that the absolute strengths by the two types of test are practically equal. From a theoretical consideration of the two types of test specimens one would expect the mortar cubes to show apparent compressive strengths higher than those shown by the concrete cylinders. Using Mr. Moore's prescribed method we made mortar cube specimens with the first seven of the eight cements of the recent cooperative investigation conducted by the Society's Committee C-1 on Cement.[1] The strengths as shown by these cubes were compared with the concrete strengths obtained by our laboratory in the investigation. At two days the mortar strength for the lowest testing cement was 82 per cent of the concrete strength; for the highest testing cement the mortar strength was 123 per cent of the concrete strength; and the average mortar strength was 108 per cent of the concrete strength. At 4 days the mortar strengths ranged from 106 per cent to 138 per cent of the concrete strengths and the average was 118 per cent. At seven days the range was from 103 to 139 and the average was 120 per cent. We did not get the close agreement in absolute strengths that is shown by Mr. Moore in Figs. 6 and 7 in his paper.

Despite these apparently undesirable features in the standard sand mortar it is interesting to note that when the mortars and concretes were both computed on a relative strength basis the standard sand mortar showed about the same order of agreement with the concrete as was shown by the local sand mortar cubes and the run-of-mine Ottawa sand mortar cubes which were included in the original cooperative investigation.

In the last part of his paper Mr. Moore proposed a method for the predetermination of compressive strength by means of an earlier test of a mortar with a lower water-cement ratio. The method is evidently based on a series of strength-age curves derived by averaging the test results for a considerable number of cements. There may be some question as to whether the strength-age curves for individual cements in general are sufficiently close to the average curves shown by Mr. Moore to warrant the use of his method without further knowledge of the cement to be tested in this manner. We have encountered cements with strength-age curves sufficiently subnormal or abnormal so that predetermined strengths might be as much as 25 per cent above or below the actual strengths. It is quite possible that the disagreements between actual and predetermined strengths in Fig. 9 are due to strength-age characteristics in the respective cements and not to any lack of control in laboratory procedure.

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[1] Report of Committee C-1 on Cement, *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part I, p. 258 (1929).

MR. CLOYD M. CHAPMAN.[1]—I wish merely to emphasize, to say over again what I have said many times before, that investigations of the sort Mr. Moore is engaged in are bound to be helpful, whether they prove to be on the right track or simply succeed in eliminating one more wrong way to do a thing. I am very glad to know that the Central Laboratory has tried to check this method, and I hope that a dozen other laboratories will try to check it. If the method is right, or is wrong, the earlier we find it out, the better off we will be. I trust there is sufficient evidence presented in Mr. Moore's paper that there may be enough of value in the method to stimulate a dozen other laboratories to try it out just as the Central Laboratory has done.

MR. H. F. GONNERMAN[2] (*presented in written form*).—The author deserves much credit for presenting to the Society a great amount of valuable information on tests of concrete and mortars. The tests have been planned and carried out with great care and the results are quite uniform throughout the range covered.

One of Mr. Moore's most noteworthy findings is the close agreement between strength and water-cement ratio for a large range in sizes of aggregate. His Fig. 1 shows that for a given water-cement ratio the compressive strength of mortar containing 0 to No. 100 sand is just about equal to the compressive strength of concrete made with 1½-in. maximum size aggregate. The standard sand specimens, however, show considerably lower strengths than the others for given water-cement ratios. The lower strengths are generally attributed to the lack of workability of the standard sand mortars.[3] The uniformly-sized particles produce harsh mortars which do not meet the ordinary requirements of workability. If this harshness can be eliminated by using a different gradation of the silica sand, better concordance in results between the mortar and concrete strengths would be obtained.

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[1] Consulting Engineer, New York City.

[2] Manager, Research Laboratory, Portland Cement Assn., Chicago, Ill.

[3] See Report of the Director of Research, Portland Cement Assn., 1928, p. 42.

Ottawa silica sand has desirable properties for cement testing in that it has practically no absorption and is composed of structurally sound grains of uniform shape and composition. Mr. Moore has made a commendable attempt to reduce the effect of harshness of standard sand mortars by making comparisons of mortars and concretes from graded aggregates of given water-cement ratios with standard Ottawa sand mortars having void-cement ratios equal to the water-cement ratios of the concretes and the graded natural sand mortars. The determination of the correct void-cement ratio involves a number of trial batches and is, therefore, a rather tedious procedure. It is also subject to variation due to the personal equation of the operator and inaccuracies of measurement. Some idea of the difficulties involved is illustrated in Mr. Moore's paper which shows that for a number of concrete mixes, the void-cement ratio was found to be as much as 7½ per cent less than the water-cement ratio. Since the amount of cement remained constant the sum of the water and air voids was found to be 7½ per cent less than the water voids alone, which, of course, is impossible. This is probably due to the fact that

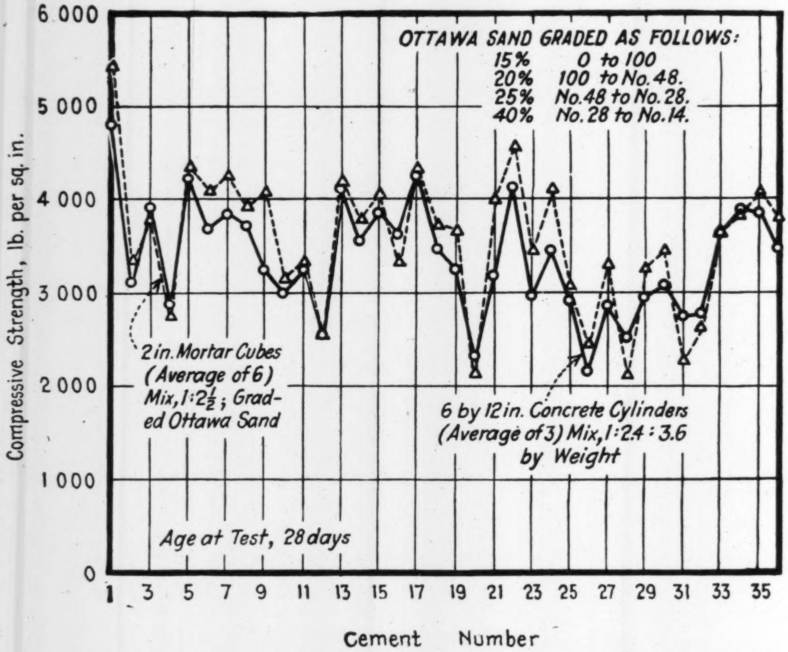

FIG. 1.—Compressive Strength of Concrete and Plastic Graded Ottawa Sand Mortar from 36 Portland Cements.

no correction was made for water absorbed by the aggregate when using apparent specific gravity.

Tests show that practically no air voids exist in a plastic concrete mix. The void-cement ratios, therefore, are identical with water-cement ratios for such mixes. While the determination of the actual voids in a mix is a rather complicated operation, the water-cement ratio can be accurately determined and is, therefore, more readily adapted to cement testing.

Much work along the same lines as Mr. Moore's has been carried out at the laboratory of the Portland Cement Association as mentioned in the speaker's discussion of Mr. Brickett's paper at the 1928 meeting of this Society.[1] Tests were made on 2-in. mortar cubes and concrete of 0.90 net water-cement ratio. The sand for the mortar was 0 to No. 14 silica sand from Ottawa, Ill., from the same source as standard Ottawa sand. When mixed in the proportions of 1:2.5 by weight with a water-cement ratio of 0.90, the mortar was of such consistency that it could be readily molded without segregation. The 28-day strengths of these cubes are compared with the strengths of 6 by 12-in. concrete cylinders of the same water-cement ratio in the accompanying Fig. 1 which brings out the agreement between the mortar and concrete strengths for the 36 cements tested. In general, the cubes show slightly higher strengths than the cylinders at this age.

The speaker believes that a graded Ottawa sand has advantages over standard Ottawa sand for standard compression tests of cement since it possesses all the desirable features of the standard Ottawa sand and at the same time gives mortar strengths equal to those of concrete of the same net water-cement ratio.

Some of the relationships Mr. Brickett described were brought out in a recent progress report of Sub-Committee VII of Committee C-1 on Cement where it was shown that the mortar cube strength at the early ages seemed to run a little lower than the compressive strength of the concrete. However, as the age of the material increased, or the strength of the cement was higher, the cube strength tended to run higher than the concrete strength. In other words, there is a curvilinear instead of a straight line relationship between cube and concrete strength. Our laboratory is carrying out tests along the same lines indicated by Mr. Moore, and we are planning to make tests of these mortars in 2-in. cubes as well as in 6 by 12-in. cylinders in order to determine the relationship between the cube and cylinder strengths.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 445 (1928).

MR. P. H. BATES.[1]—We tried this method in the Bureau of Standards and did not get such concordance as we thought we should, judging from Mr. Moore's work. But on the other hand, we did not obtain concordance with the Brickett method at first. We felt that considerably more work should be done because we were "green" and possibly some of these laboratories that have tried it are equally green. If they continue their operations, possibly they may obtain as good results as Mr. Moore does in his laboratory. It is worth considerable further thought.

MR. J. C. PEARSON.[2]—I should like to ask whether Mr. Moore has any theory to account for the apparently high air voids in the Ottawa sand mortar? When one considers the roundness and smoothness of the Ottawa sand grains, one would expect less air to be entrapped in mortar made with this sand than in ordinary mortar, made with sand of rough texture and more or less pitted.

MR. CHAPMAN.—Before Mr. Moore answers that question, I should like to know who believes the statement that the amount of air in soft mortar or mortar of low consistency is small? There is a lot of air in any mortar that I ever tried to take the air out of, and if you will just take a bell jar and mix up a glass full of mortar by any method you can devise, and put it under the bell jar and exhaust the air from it, you will get, at about 15 to 20 in. of vacuum, enough air out of that mortar to surprise yourself. The mortar may be mixed by any method of mixing you can devise.

MR. PEARSON.—I think Mr. Chapman misunderstood the statement. Mr. Moore has brought out the fact that there is a lot more air in Ottawa sand mortar than in mortar made with other sands. What is the explanation?

MR. CHAPMAN.—I understood Mr. Gonnerman to say that the amount of air ought to be small.

MR. GONNERMAN.—We tested several mixtures of standard Ottawa sand with the following results:

WATER-CEMENT RATIO		VOID-CEMENT RATIO, BY VOLUME	AIR VOIDS, PER CENT BY VOLUME OF MORTAR
GALLONS PER SACK	VOLUMETRIC RATIO		
4	0.534	0.604	3.5
5	0.667	0.833	6.2
6	0.800	1.024	7.0
7	0.933	1.226	7.8
8	1.067	1.355	7.3

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[1] Chief, Clay and Silicate Products Division, U. S. Bureau of Standards, Chevy Chase, Washington, D. C.

[2] Assistant to Chemical Engineer, Lehigh Portland Cement Co., Allentown, Pa.

In other tests Ottawa silica sand was used in which there was considerable fine material, and for several sand mixtures of that type, in a 1:1.6 mix by weight there was about 0.5 per cent air voids in the mortar. These figures will give some idea of the air voids in different mixtures of standard Ottawa and graded Ottawa sand.

In our laboratory we have a special device for measuring the air voids, which consists of a steel pipe 6 in. in inside diameter and about 18 in. long which is welded to a bottom plate. The volume of material in this container is obtained by measuring the distance from the top of the container to the surface of the material by means of an accurately-graduated calibrated bar. We checked up on Mr. Moore's curve and obtained the same values he did at the end points but the intermediate points deviated somewhat from the straight line relationship shown in his Fig. 4.

MR. BATES.—May I ask if Mr. Gonnerman ever extracted the air from mortar and measured it?

MR. GONNERMAN.—We have not done that. The amount of the air voids in mortars has been studied quite extensively by Talbot and Richart and in foreign laboratories. The Swiss have made a great many tests on mortars which show a considerable range in the air voids as the consistency is varied from a plastic to a very dry consistency.

MR. BATES.—I think the only method of determining the amount of air, and getting a correct value, worthy of any discussion, would be by extracting it from the mortar. It is a long difficult procedure to determine the air in any material.

MR. O. L. MOORE.[1]—In answer to Mr. Pearson's question as to why the standard sand mortar has higher air voids - cement ratio than most mortars, I do not know, but I do know that the graded or quarry run sand from Ottawa, Ill., has the same properties; that is, it has the same difference between $\frac{V}{C}$ and $\frac{W}{C}$, which indicates that it is not due to the uniform grading of standard sand. It may be due to a surface condition of the particles.

(*Author's closure by letter*).—With the several contributions to the discussion in mind we should like to emphasize again the principal feature that the compressive strength of plastic standard sand mortar obeys the voids-cement ratio - strength relation which offers a sound basis for a method of test with a sand of practically constant properties. It is recognized that the voids-cement ratio - compressive strength curve of plastic sand mortar and that of workable concrete coincide in the higher ratios but separate as the ratios decrease, the mortar testing above the concrete. For this reason a limit of application to a practical range as determined by our experience was suggested. This separation of the curves in the region of low ratios has received considerable investigation in the past and it merits further investigation. It is reasonable to expect that the voids-cement ratio - strength relations of mortar and concrete coincide throughout the entire range of practical ratios and that such factors as the standard method of loading compression test specimens may account for the divergence. We are undertaking an investigation along this line at the present time.

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[1] Engineer of Tests, Universal Portland Cement Co., Chicago, Ill.

THE TENSILE AUTOGENOUS HEALING OF PORTLAND-CEMENT MIXTURES

By Herbert J. Gilkey[1]

Synopsis

In a former paper[2] there were recorded considerable data upon compressive autogenous healing of portland-cement mortars and concretes. It was evident that, if compressive healing occurred, there must be tensile healing, since tensile, compressive and shearing resistance are, in general, but varied manifestations of the same thing. While a preliminary test referred to in the earlier paper failed to produce evidence of it, subsequent experiments have shown that tensile autogenous healing is a reality for mixtures of portland cement and water, with or without aggregate of usual accepted kinds. Healing occurs under the same conditions as initial moist curing and is essentially but a continuation or a resumption of the normal curing processes. It is evidently not in the least due to redeposition of soluble salts in cracks, as has been sometimes supposed. Standard tensile briquets of widely varying water-cement and cement-sand ratios (from 0.20 to 1.20 and from neat cement to 1:5, respectively) have been completely severed as many as six times at initial test ages of from 3 days to 27 months. All mixtures and conditions have shown measurable healing. Individual recoveries range from zero to 221 lb. at 3.5 months for a neat cement specimen initially testing 248 lb. at 3 days. Severed fragments must be closely fitted, but the amount of pressure between the halves appears to be unimportant if the crack be kept closed. Healing will occur upon re-immersion after months of drying out. It occurs in either running or stagnant water. In general the extent of healing is less after each successive retest, although there were many individual exceptions. The majority of the specimens gave no strength at the fifth or sixth breaking, although many carried a few pounds.

Knowledge of tensile healing is of practical use in various connections and in addition it should contribute to a better understanding of concrete. The healing is not related to an interesting form of crack closure that often appears in sidewalks. This crack closure may be related to the surface disappearance of cracks in air-exposed concrete as noted by Abrams, Earley and Hollister.

Introduction

The status of compressive autogenous healing was rather fully outlined in a former paper on the subject.[2] Other somewhat incidental reports of similar observations were given in a prior publication.[1] Apparently no mention of the phenomenon of direct tensile healing of fully severed parts is made in the technical publications other than the reports by the present author[2] on parts of the work of which this paper is a record. There are a number of references to healing and crack closures in partially severed members such as beams.[3]

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[1] Associate Professor of Civil Engineering, University of Colorado, Boulder, Colo.

[2] H. J. Gilkey, "The Autogenous Healing of Concretes and Mortars," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 470 (1926).

Since the strength of concrete is largely due to bond or adhesion between particles, it is evident that strength may be manifested as compressive, tensile, or shearing resistance to load. Thus if there is compressive or flexural healing, obviously there must be tensile healing. The first attempt to verify this gave a negative result as mentioned in the previous paper, but all of the later tests, some of which this paper records, amply demonstrate the reality of tensile self-healing.

In all compressive strength manifestations, direct normal bearing and internal friction or sliding bond join with the direct adhesion to develop a relatively high resistance for a very slight adhesive increment. In tension tests the adhesion is practically unaided and in this respect the tension test is a much fairer measure of that which gives strength to the concrete or mortar. As would be expected, then, the percentage of tensile strength recovery should be much less than that in compression, and this expectation is supported by the tests.

Like the former investigation the autogenous healing results are really by-products of other projects. In all cases the specimens were standard briquets, such as are specified in the standard tests for cement. Broken briquet halves that were usually thrown away were saved for healing tests.

The healing is evidently due to fresh crystallin growth resulting from the resumed or continued normal curing processes rather than to any redesposition of soluble salts, as was suggested in the former paper.

MATERIALS

All of these tests were on portland-cement mortars, but the conclusions reached are applicable by analogy to concrete, since the size of aggregate is the sole difference between the two. This is a non-relevant factor so far as the nature of the action is concerned.

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[1] *Proceedings*, Am. Concrete Inst., Vol. XXII, Table 9, p. 406 (1926).

[2] Discussion of Report of Committee C-1 on Cement, *Proceedings*, Am Soc. Testing Mats., Vol. 28, Part I, pp. 304-305 (1928).

[3] D. A. Abrams, "Test of a 40-foot Reinforced Concrete Highway Bridge," *Proceedings*, Am. Soc. Testing Mats., Vol. XIII, p. 884 (1913); also Abrams, Earley and Hollister, *Proceedings*, Am. Concrete Inst., Vol. XXII, pp. 636–639 (1926).

The tests include many representative brands of portland cement and preliminary tests not here recorded included several local natural sands, both fine and coarse, but all passing the No. 4 sieve. These were creek sands with rounded particles of many different compositions, but with quartz grains predominating. A great many of the tests were with standard Ottawa sand of the No. 20 to No. 30 size. Sand mortars were in the proportion 1:3 by weight, except for one extensive series which includes mortars ranging from the neat cement to 1:5 cement-Ottawa sand mixtures. These mixtures covered a wide range of consistency, extending from the very wet to the unworkably dry for most of the proportions.

TEST METHODS

As mentioned, all specimens were standard tensile briquets made in the regular molds with a cross-sectional area of 1 sq. in. Standard procedure was followed in making, storing and testing until after the original, or initial, breaking. All tests were made on a 2000-lb. hand operated Riehlé lever briquet testing machine.

After each breaking, the two halves were refitted as perfectly as possible and a rubber band (in a few cases a thread was used) was snapped around the specimen, which was re-immersed in water and left undisturbed until the next test.

The size of rubber bands was varied, and rubber bands were also compared with tightly stretched threads in some of the early series. These seemed to be irrelevant variables so long as the fitting was perfect enough and pressure sufficient to keep the halves in close contact during the healing period.

To check the redeposition theory one preliminary series was divided after the first breaking and alternate specimens were stored in stagnant water in which many specimens had cured and which was therefore heavily charged with calcium hydroxide. The others were re-stored in a large tank in which the water was kept fresh by constant inlet and an overflow. The circulation was slow but continuous. Healing to about the same individual maximum amounts occurred under both storages. The number of specimens was too few to furnish any proper basis for a quantitative conclusion, but the important point is that healing did take place in both environments and it does not appear, therefore, that the condition of the water can be rated as a major factor. None of the data of these tests appear in this paper.

Another preliminary observation which is mainly of qualitative significance was the fact that specimens, the halves of which remained separated for 9.5 months in dry air on a shelf over a radiator (hot much of the time) healed to what appeared to be about the same extent, as if they had never dried out, when refitted and re-immersed in water for 3 months. Healing itself never occurs in the absence of moisture, but a period of dryness seems to be only one of suspended healing activity, which parallels the general curing phenomenon.

Other miscellaneous observations for which no data are included are:

1. Natural sand briquets of a 1:2.5 mix by weight, water-cement ratio 0.70, initially broken at 27 months gave retest strengths of from one to three pounds 12 days later. Moreover, some of these old specimens were definitely stuck together (as described below) although all had been separated and weighed less than nine months before.

2. Stacks of briquets (especially neat ones because of the very smooth surfaces) often adhere so firmly that the top briquet will lift several others and a screw driver or other sharp-edged pry may be necessary to separate them. The healed areas that were in contact exhibit the same dull texture and crystal faces as are observed on the interior of a healed specimen after retest. It is not essential that the specimens in the stack be of the same cement.

3. Two by four-inch cylinders stored side by side have adhered along the element of tangency so firmly that one would lift the other.

In retesting, the specimen was carefully picked up and pulled slightly by hand. Sometimes there was no healing, in which case the two halves separated as soon as the band was removed. In other cases there was only a slight sticking and the load was recorded as 0+. The lower grip on the testing machine weighs about 3 lb. If the specimen pulled apart as the weight of the grip came on it, the load was recorded as 3−. If it fell apart after carrying the weight of the grip, but before additional measurable load had been applied, the load was recorded as 3 or 3+. Loads were applied slowly and continuously to failure. Loads above 3 lb. are test loads as read on the beam of the machine.

TEST DATA

The tables and curves supply the record of numerical data. Details of mixtures, materials, variables, etc., accompany the tabular and graphical presentations. The data proper will be discussed in connection with these. There are, however, certain general observations in the nature of descriptive data that supply needed background for the more correct evaluation of what is to follow.

Generally speaking the healed area was easily recognized after the specimen was pulled apart. The fracture was always coincident with that from the original test, since the recovery never approximated the strength of the unbroken material. The parts that had healed were dull and moist in color, except for many of the neat cement specimens which showed distinct crystallin structure and many bright crystal faces. Those most in evidence to the naked eye were rectangular and were about twice as long as wide. The longest were possibly $\frac{1}{16}$ in., or a trifle more, in length.

All parts of the area that did not heal were covered with the shiny water film of any wetted surface.

Healing was never observed to be present over an entire fractured area. The outside margin for $\frac{1}{8}$ in. or so rarely, if ever, gave any evidence of healing. Often 50 per cent or more of the surface showed healing. Sometimes there were separated healed areas, but more often these areas appeared to be contiguous.

Healing never occurred across a crack of appreciable width, and a misplaced sand grain precluded healing except over a very limited territory. On the other hand, it was not necessary that the two halves be perfectly matched laterally. Smoothly fractured, neat cement specimens were apparently unaffected by lateral displacement of $\frac{1}{8}$ in. or more. This is more clearly evidenced by the tenacity with which stacks of briquets often cling to one another in a curing tank as was previously mentioned.

There was never any evidence of the disappearance of a crack. It was often possible to re-fit the halves so perfectly that a crack was scarcely visible; but there was no indication that the crack was less visible at a later date. Moreover, the apparent absence of healing around the outer margin seems to indicate no crack closure and that "redeposition of soluble salts" does not explain the healing action. Redeposition may explain the disappearance of the crack, as noted later.

In all of the work devoted to attempts to evaluate the extent of recovery or self-healing there have been present certain intangible, and apparently rather insurmountable difficulties. Thus in connection with the compressive autogenous healing, previously cited, it was impossible to appraise the initial damage with any degree of precision. The reference datum was probably the most indeterminate factor present. In tensile healing this difficulty is completely overcome. Every test completely severed the briquet and the specimen starts healing from a state of zero residual tensile strength. There are present, nevertheless, at least three factors that are probably disturbing enough to make it impossible to establish any satisfying quantitative laws; certainly such laws are not yet apparent. Perhaps a different approach or an improved technique may yet enable someone

TABLE I.—TENSILE HEALING FOR VARIABLE PROPORTIONS OF CEMENT, STANDARD OTTAWA SAND, AND WATER.

(See Also Figs. 1 and 2.)

All specimens are briquets of standard form. Proportions of cement and sand are by weight. Water-cement ratios are by loose volume, the range from 0.20 to 1.20 = 0.13 to 0.80 by weight or 1.50 to 9.00 gal. per bag cement. The values in each line are the means of 3 test results, except lines 5, 6, 7, 17, 24 which are each the mean of 6 results.

	Mix	Tensile Strength, lb. per sq. in.							Tensile Strength at 7 Days, lb. per sq. in.	Compressive Strength at 28 Days, lb. per sq. in.	Unit Weight, lb. per cu. ft.	Density (Solidity Ratio)	Ratio of Water to Total Voids
		Water-Cement Ratio $\left(\frac{W}{C}\right)$	Original Test, Age 28 days	Retest No. 1, Age 6 months, 5 months since last test	Retest No. 2, Age 9 months, 3 months since last test	Retest No. 3, Age 1 year, 3 months since last test	Retest No. 4, Age 1.5 years, 6 months since last test						
	1	*2*	*3*	*4*	*5*	*6*	*7*		*8*	*9*	*10*	*11*	*12*
1	Neat*	0.20	411	40 (88)	6 (14)	5 (8)	1 (3)		187	3853	116.1	0.52	0.45
2	Neat*	0.25	602	43 (58)	8 (12)	2 (6)	4 (6)		324	4630	125.8	0.55	0.63
3	Neat*	0.30	898	64 (72)	2 (5)	0 (0+)	0 (0)		852	9390	133.7	0.57	0.82
4	**Neat***	**0.33**	**910**	Maximum Mixtures Interpolated from Fig. 1.					**860**	**9700**	.....	....	....
5	Neat	0.40[a]	761	34 (66)	4 (12)	0 (0+)	0 (0)		676	9110	131.6	0.53	0.94
6	Neat	0.60[a]	569	21 (53)	4 (23)	1 (3)	1 (3)		449	6536	121.9	0.44	1.00
7	Neat	0.80[a]	329	34 (124)	8 (33)	1 (3+)	2 (8)		234	3500	113.1	0.38	1.00
8	1:1*	0.25	448	14 (22)	2 (5)	2 (7)	1 (3)		311	3817	133.2	0.68	0.52
9	1:1*	0.30	539	20 (25)	6 (8)	4 (4)	4 (5)		382	4880	144.1	0.73	0.77
10	1:1*	0.35	738	42 (48)	2 (3+)	1 (3)	0 (0+)		700	7927	143.1	0.71	0.83
11	**1:1**	**0.39**	**780**	Maximum Mixtures Interpolated from Fig. 1.					**790**	**8750**	.....	....	....
12	1:1	0.40	764	14 (21)	8 (12)	3 (3+)	1 (3+)		781	8700	143.4	0.71	0.91
13	1:1	0.60	634	9 (25)	1 (4)	1 (3+)	0 (0+)		492	6770	136.9	0.64	1.00
14	1:1	0.80	550	30 (33)	5 (9)	2 (4)	1 (3)		404	4757	130.9	0.58	1.00
15	1:1.5*	0.40	709	17 (27)	1 (3)	2 (3)	3 (5)		618	6980	144.7	0.74	0.86
16	**1:1.5**	**0.43**	**720**	Maximum Mixtures Interpolated from Fig. 1.					**630**	**7200**	.....	....	....
17	1:1.5	0.80[a]	547	21 (45)	8 (12)	2 (3)	1 (3)		411	4192	135.2	0.63	1.00
18	1:2.5*	0.35	326	20 (25)	1 (3)	1 (3)	2 (3)		276	3045	138.0	0.75	0.54
19	1:2.5*	0.40	219	10 (13)	1 (3)	0 (0)	0 (0)		188	4784	143.8	0.77	0.72
20	**1:2.5**	**0.48**	**560**	Maximum Mixtures Interpolated from Fig. 1.					**460**	**5450**	.....	....	....
21	1:2.5	0.60	539	8 (21)	2 (3+)	1 (3)	0 (0)		388	4270	139.4	0.72	0.82
22	1:2.5	0.80	452	38 (38)	5 (10)	4 (7)	8 (9)		312	3222	137.0	0.69	0.92
23	1:3*	0.40	198	8 (23)	3 (9)	1 (3)	0 (0)		160	1450	132.2	0.72	0.47
24	1:3*	0.50[a]	396	11 (28)	1 (8)	1 (3)	3 (9)		302	2580	137.5	0.74	0.63
25	**1:3**	**0.54**	**420**	Maximum Mixtures Interpolated from Fig. 1.					**330**	**3300**	.....	....	....
26	1:3	0.60	411	18 (26)	4 (7)	0 (0)	0 (0)		305	3250	140.6	0.74	0.79
27	1:3	0.70	422	31 (35)	0 (0+)	0 (0)	0 (0)		328	3190	140.0	0.73	0.86
28	1:3	0.80	379	11 (12)	1 (3+)	0 (0)	1 (3)		252	3067	134.1	0.69	0.81
29	1:3.5*	0.40	207	6 (11)	2 (4)	1 (3)	3 (9)		140	1650	128.2	0.71	0.39
30	1:3.5*	0.60	313	6 (17)	0 (0+)	0 (0)	1 (3)		228	2050	137.5	0.74	0.68
31	1:3.5*	0 65	359	15 (34)	5 (13)	0 (0+)	3 (6)		300	2770	137.5	0.73	0.72
32	**1:3.5**	**0.68**	**370**	Maximum Mixtures Interpolated from Fig. 1.					**300**	**2800**	.....	....	....
33	1:3.5	0.80	331	1 (3+)	0 (0+)	0 (0)	0(0)		221	2393	136.2	0.71	0.80
34	1:4.5*	0.60	200	2 (5)	0 (0+)	0 (0+)	0 (0)		148	1418	129.7	0.71	0.48
35	1:4.5*	0.70	270	13 (26)	4 (11)	1 (3)	2 (5)		180	1935	132.9	0.72	0.59
36	**1:4.5**	**0.74**	**285**	Maximum Mixtures Interpolated from Fig. 1.					**190**	**2000**	.....	....	....
37	1:4.5	0.80	268	18 (26)	5 (11)	3 (5)	0 (0+)		183	1780	132.0	0.71	0.64
38	1:4.5	1.20	169	0 (0)	0 (0)	0 (0)	0 (0)		118	1039	133.0	0.68	0.85
39	1:5*	0.60	180	5 (15)	0 (0+)	0 (0+)	2 (7)		128	1155	126.4	0.70	0.42
40	1:5*	0.80	212	14 (36)	3 (5)	1 (3)	0 (0+)		145	1296	129.5	0.70	0.56
41	**1:5**	**0.85**	**215**	Maximum Mixtures Interpolated from Fig. 1.					**160**	**1300**	.....	....	....
42	1:5	0.90	204	26 (36)	4 (9)	2 (4)	0 (0+)		156	1337	130.8	0.70	0.63
43	1:5	1.00	184	14 (38)	0 (0)	0 (0)	3 (6)		121	1154	137.6	0.70	0.71

[a] Value is the mean of 6 results.

NOTE.—After each average retest value (columns 4 to 7) is given the highest **single retest of** the group. This maximum retest value is probably much more significant than the average retest **strength. Starred mixtures** are on the dry side **of workability.** Probable maximum strength mixture for each **sand-cement ratio is interpolated** from Fig. 1 **and appears in boldface type. Individual specimens that gave high initial or retest strengths were not necessarily the highest at subsequent tests.**

to bridge the gap. Qualitatively a great deal of a definite nature has been learned.

The three difficult variables alluded to are:

1. The effect of perfection or imperfection of fit when the two broken halves are matched. Misplaced sand grains, etc., make the "crack variable" one that is often present but difficult to evaluate.

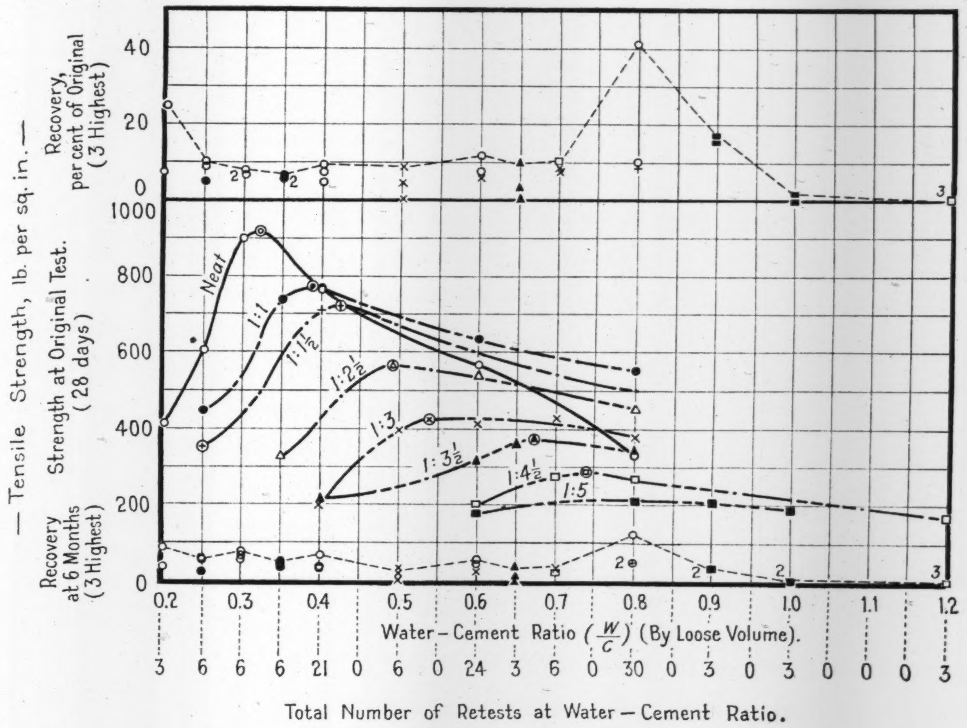

FIG. 1.—28-Day Original and 6-Months Retest Strengths of Cement-Standard Ottawa Sand Mixtures at Various Water-Cement Ratios.

See Table I for detailed data. Proportions are by weight. Each plotted point represents the mean of from 3 to 6 tests (except circled points which are only approximate locations). Each retest point represents an individual retest strength. In general only the 3 highest retests are shown.

2. The entire area does not heal. The outer margin appears to be immune and the mending, or crystal growth, seems to start at some interior point or points from which it probably spreads. Just what factors may determine the area to be healed is not now apparent. The "crack variable" mentioned above is doubtless the most important single one but there are probably others as well.

3. Eccentricity of resistance. As has been pointed out, the healing usually occurs in the interior, but it is not necessarily at the center of the cross-section. Thus the load and the area that resists load are not collinear and the stress induced is a combination of direct stress and bending which causes a progressive tearing failure with a lower maximum load carried.

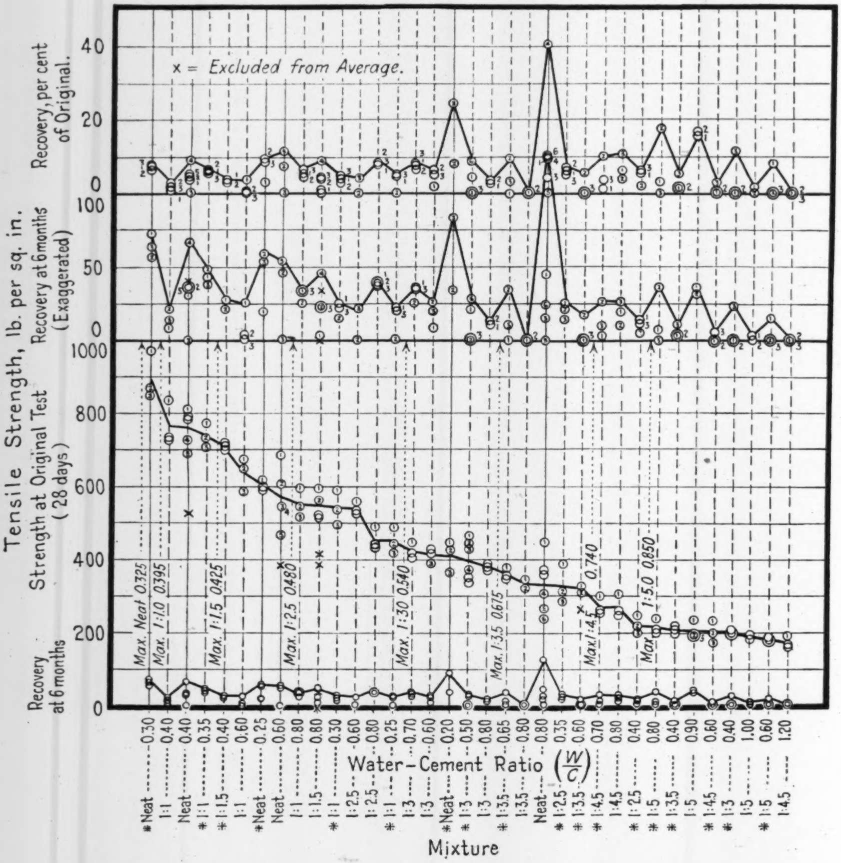

FIG. 2.—Strength Recovery at 6 Months Compared with Average Strengths at Original Test (28 days). Mixtures are arranged in order of average 28-day strength at original test.

See Table I for detailed data. All mixtures are cement and standard Ottawa sand proportioned by weight.

All plotted points are individual tests or retests. The numbers in the circles indicate the strongest, next strongest, etc., of group at original test. Average curve is shown for original 28-day tests. Only upper points are connected for retest curves.

Maximum strength mixtures are interpolated from Fig. 1. Starred values are for water-cement ratios below those for maximum strength (too dry for good workability).

Because of these three uncertain factors, companion specimens subjected to identical treatments are unlikely to show agreement with one another in healing strengths attained. It therefore appears that average healing performance is less significant than individual. Moreover all the recognized disturbing factors are such as to reduce the healed strength and it therefore seems logical to focus the attention upon the strongest of the retests of a companion group and to weigh rather lightly the average or weaker results. This presupposes, of course, that the major objective is to ascertain just what degree of healing is possible under normally favorable conditions. Even when the highest retest strengths are considered, it is safe to assume that they are below what they might have been had the unfavorable factors been present to a still less degree.

DISCUSSION OF TEST DATA

Table I supplies the data for series No. 4, the most searching of the several series conducted. This series was from one cement used neat or with from one to five parts by weight of standard Ottawa sand (No. 20 to No. 30 grading). Consistencies were varied from the very fluid to the unworkably stiff for the different cement and sand combinations. Figures 1 and 2 apply to this series.

Figure 1 shows the 28-day tensile strength - water-cement ratio curves and the actual recovery at 6 months below and percentages (in terms of 28-day strength) above. For a given water-cement ratio, in general, only the three highest retest strengths are plotted, regardless of the mixture. No systematic healing relationship is in evidence from Fig. 1.

Figure 2 shows a more or less crudely defined healing law. While the actual strength at the first retest decreases somewhat as the strength at original test becomes less (mixture leaner or wetter, or else too dry) the percentage healing increases. That is, the weaker mixtures or specimens recover a less amount but a relatively higher percentage of their initial strengths. Such action is not very positively indicated. In Fig. 2 the mixtures on the dry side of workability are starred to facilitate comparison on the basis of workable mixtures only, if preferred. Probable mean maximum strength mixtures for each sand-cement ratio are interpolated from Fig. 1 or Table I.

Neither of the figures deals with retests other than the first. Table I supplies the numerical record and will bear study. Even at the fifth breaking (fourth retest) a specimen as lean as 1:4.5 gave a strength of 5 lb. after having shown no strength at the fourth breaking, and 26 lb. at the first retest. This specimen was the weakest of three companion specimens at original 28-day test, but was the strongest at the first and fourth retests, which only serves to illustrate the difficulty encountered in attempting to generalize from the retest data. In Table I all mixtures on the dry side of workability are starred. Maximum strength mixtures (boldfaced type) are interpolated values from the curves of Fig. 1.

While not directly related to the healing phenomenon, the data of columns 8 to 12 should assist greatly in visualizing the mixtures. From them one can make some very interesting comparisons between 7 and 28-day tensile strengths and 28-day compressive strengths. Unit weights and densities (in the sense of solidity ratios) may be studied, and column 12 is very instructive in showing what portion of the voids in the mixtures (as placed) was occupied by water.[1,2,3]

Data from Cooperative Tests of Committee C-1 on Cement:

Tables II and III (Plates II and III) were abstracted in the former discussion.[4] While detailed consideration is inadvisable it is worth while to note the extent to which certain cements appear among those giving maximum values of tests or retests (extreme right-hand columns). As usual, the only positive deduction is that healing of considerable magnitude occurred both for the neat cements of Table II and the 1:3 mixtures of Table III. What may be trends have so many individual exceptions that it is difficult to generalize. Some of the cements consistently rate high in healing performance at several retest ages and for both mixtures. Others are erratic.

Using Table II as illustrative, and bearing in mind that original tests were made at 3, 7, 28 days and 3 months for specimens Nos. 1 to 3, 4 to 6, 7 to 9, and 10 to 12, respectively, and that the first retests were made at a total age of 3.5 months for all specimens, the following facts are apparent but their interpretation is inconclusive.

Generally speaking the first retest strengths are greatest for the specimens initially broken at 3 days, and they follow the order of 3, 7, 28 days and 3 months, the maximum values being 221, 163, 57 and 33 lb., respectively, as shown in lines 13, 27, 36, and 55. For the 1:3 mixtures (Table III) the corresponding maximum retest strengths are 143, 53, 27 and 4 lb., respectively. In attempting to

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[1] These data will bear study and analysis in connection with the treatment of ***Bulletin 137***, Engineering Experiment Station, University of Illinois (Talbot and Richart). Similar data appear in Fig. 17, p. 45 of the ***Bulletin***.

[2] See also ***Proceedings***, Am. Concrete Inst., Vol. XXII, Table 2, Columns *l, m, n* (1926).

[3] See also ***Proceedings***, Am. Concrete Inst., Vol. XXIII, Figs. 3 and 4, pp. 373-374 (1927).

[4] ***Proceedings***, Am. Soc. Testing Mats., Vol 28, Part I, pp. 304-305 (1928).

TABLE II.—ORIGINAL AND RETEST TENSILE STRE

Water, 42 per cent by weight.
All specimens made and originally tested as part of the 1927 cooperati
See *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 261 (1928

	Specimen Number	Total Age	Time Elapsed Since Last Break	Break Number	Retest Number	Cemen															
						1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1	1	3 days......	....	1	..	280	203	222	145	240	188	193	191	142	110	136	189	154	135	223	183
2		3.5 months	3.4 months	2	1	78[?]	[c]30	[c]33	108	46	74	90	102	[d]74	48	44	186[?]	130	[c]11	103	70
3		6.7 months	3.2 months	3	2	0	0+	16	27	4	0	17	0	0	0+	0+	0	10[?]	0	11	0-
4		9.0 months	2.3 months	4	3	0+	0	0+	10	0+	0	0+	0+	0+	0	0	0+	0+	0	..	..
5		15.0 months	6.0 months	5	4	5	0+	3	11	4	0+	0+	5	0+	0+	0	0	3-	0+	..	..
6		15.0 months	8.3 months	4	3	..	..	..	..	..	..	..	..	..	..	..	..	..	..	5	0-
7	2	3 days......	....	1	..	300	211	222	149	262	191	216	201	148	118	133	192	153	135	221	173
8		3.5 months	3.4 months	2	1	79	55	135	[c]40	63[?]	113	57	115	141	67	105	164[?]	[c]112	29	[c]72	71
9		9.0 months	5.5 months	3	2	12	23	3+	0	3-	11	10	10	4	11	3+	3-	11	0+	..	..
10		15.0 months	6.0 months	4	3	0	3-	0+	0	0	0	0	0+	0	0	0	0+	0	0	..	..
11		15.0 months	11.5 months	3	2	..	..	..	..	..	..	..	..	..	..	..	..	..	..	22	22
12	3	3 days......	....	1	..	248	170	198	158	222	193	[a]173	196	157	121	125	[a]122	171	129	209	166
13		3.5 months	3.4 months	2	1	221	55	82	56	51	154	132[?]	142	124	[d]33	91	47	[c]138	53	[c]3+	[d]3+
14		15.0 months	11.5 months	3	2	59	42	22	21	9	41	3+	8	30	5	13	4	17	5	..	..
15	4	7 days......	....	1	..	392	312	283	222	354	309	327	242	263	229	246	212	265	234	322	304
16		3.5 months	3.3 months	2	1	64	[d]8	[c]46	80	[d]14	110	68	113	60	108	[d]0+	114[?]	133	90	77	3+
17		6.7 months	3.2 months	3	2	0+	0+	3-	0+	0+	14	14	14	3-	16	..	0+	22	8	8	0
18		9.0 months	2.3 months	4	3	0	0	0	0+	0+	0+	0+	0+	0	0+	0	0+	3-	0+	..	..
19		15.0 months	6.0 months	5	4	0	3	0+	3-	3+	0+	3-	3	0+	3-	0	0+	3	3	..	..
20		15.0 months	8.3 months	4	3	..	..	..	..	..	..	..	..	..	..	..	..	..	..	0+	3-
21	5	7 days......	....	1	..	376	320	316	234	327	277	328	247	295	224	260	261	298	254	347	255
22		3.5 months	3.3 months	2	1	[c]24	38	99	38	78	68	..	67	79	93	[c]0+	0+	[d]0+	[c]37	[c]5	54
23		9.0 months	5.5 months	3	2	3-	3+	15	16	10	24	0	11	18	24	0+	0+	0+	3+	..	..
24		15.0 months	6.0 months	4	3	0	0	0	0	0	0	..	0	0	0+	0	0	..	0	..	..
25		15.0 months	11.5 months	3	2	..	..	..	..	..	..	..	..	..	..	..	..	..	..	0+	41
26	6	7 days......	....	1	..	387	263	307	210	301	330	339	213	285	225	266	284	263	234	361	259
27		3.5 months	3.3 months	2	1	51	[c]23	70	78	54	[d]8	[c]35	56	[c]40	0+	57	0+	163	4	..	[c]23
28		15.0 months	11.5 months	3	2	23	11	22	12	28	3+	10	30	12	0+	5	0+	37	..	..	..
29	7	28 days.....	....	1	..	621	420	477	432	570	480	493	507	535	450	561	467	496	519	588	456
30		3.5 months	2.5 months	2	1	..	27	34	20	22	[c]14	..	[c]3+	3-	15	16	12	26	3	10	[c]16
31		6.7 months	3.2 months	3	2	0	3+	3-	3+	3+	0+	0	3+	0+	0	3-	0+	0+	0+	0+	0
32		9.0 months	2.3 months	4	3	0	0	0+	0+	0+	0	0	0	0+	0+	0+	0+	0+	0+	..	..
33		15.0 months	6.0 months	5	4	0	3+	0+	3	3-	0+	..	3-	0+	0+	0	3	0+	0+	..	..
34		15.0 months	8.3 months	4	3	..	..	..	..	..	..	..	..	..	..	..	..	..	..	0+	3-
35	8	28 days.....	....	1	..	541	418	547	441	571	531	605	519	546	417	538	491	428	480	557	496
36		3.5 months	2.5 months	2	1	17	[c]0+	3+	11	33	28	24	28	13	28	3-	..	..	[c]0+	9	22
37		9.0 months	5.5 months	3	2	0	5	0+	0	13	9	0	10	0+	12	5	0	0	4	..	..
38		15.0 months	6.0 months	4	3	0	0	0	0	0	0	0	0	0	0	0	..	..	0	..	..
39		15.0 months	11.5 months	3	2	..	..	..	..	..	..	..	..	..	..	..	..	..	..	3-	3+
40	9	28 days.....	....	1	..	466	367	595	421	613	552	533	497	569	430	583	[a]358	521	477	539	438
41		3.5 months	2.5 months	2	1	23	29	0	3+	22	[c]10	15	16	15	11	4	10	..	54	18	[c]13+
42		15.0 months	11.5 months	3	2	3+[?]	33	..	5	6	..	3+	9	4	0+	0	3+	..	3+	..	..
43	10	3.0 months	....	1	..	497	527	501	597	628	534	678	676	596	516	630	513	541	585	587	496
44		3.5 months	0.5 months	2	1	19	19	14	4	5	12	0+	..	3+	0+	3+	13	7	6	11	13
45		6.7 months	3.2 months	3	2	19	48	5	16	3+	27	0	..	4	8	13	12	26	27	22	22
46		9.0 months	2.3 months	4	3	0	0+	0+	0+	0+	13	0	..	0	0	0+	4	3-	0+	..	..
47		15.0 months	6.0 months	5	4	0+	0+	0+	0	0+	3-	0	0	0+	0+	0	0	0+	3-	..	..
48		15.0 months	8.3 months	4	3	..	..	..	..	..	..	..	..	..	..	..	..	..	..	0+	3-
49	11	3.0 months	....	1	..	555	519	594	585	598	537	644	607	680	540	707	554	544	621	658	536
50		3.5 months	0.5 months	2	1	20	17	10	7	4	24	3+	3-	8	23	19	22	22	26	13	18
51		9.0 months	5.5 months	3	2	39	38	9	13	22	61	14	11	8	20	30	24	32	33	..	..
52		15.0 months	6.0 months	4	3	0	7	0	0	0+	13	0+	..	0	0	0+	3-	0	0	..	..
53		15.0 months	11.5 months	3	2	..	..	..	..	..	..	..	..	..	..	..	..	..	..	19	15
54	12	3.0 months	....	1	..	505	[a]388	569	578	[a]483	581	623	541	687	519	663	[a]427	558	667	625	557
55		3.5 months	0.5 months	2	1	10	..	12	27	12	[c]5	0+	6	25[?]	16	0	22	..	33	0+	0+
56		15.0 months	11.5 months	3	2	22	52	29	62	52	27	0	25	0+	25	..	21	..	63	..	..
57	Highest Retest........................					221	55	135	108	78	154	132[?]	142	141	108	105	186[?]	163	90	103	71
58	Second Highest Retest................					79	55	133	80	63	113	90	115	124	93	91	164[?]	138	63	77	70
59	Third Highest Retest..................					78[?]	52	99	78	54	110	68	113	79	67	57	114[?]	133	54	72	54
60	Highest (Line Number)................					13	8	8	2	22	13	13	13	8	16	8	2	27	16	2	8
61	Second Highest (Line Number)........					8	13	2	16	8	8	2	8	13	22	13	8	13	56	16	2
62	Third Highest (Line Number)..........					2	56	22	27	27	16	16	16	22	8	27	16	16	41	8	22

[a] Strength somewhat erratic. [b] Highest, second and third apply to vertical columns only. [c] Crack, broken halves not in full contact. [d] Wide crack, v

NOTE.—(0+) indicates slight adhesion up to about three pounds.

Note that specimens 1–3, 4–6, 7–9, and 10–12, respectively, are companion groups. Members of each group had identical treatment until after the fir

E STRENGTH VALUES OF NEAT CEMENT BRIQUETS.

cooperative test program under the Society's Committee C-1 on Cement.
261 (1928).

Cements 1-32																	Highest	Second Highest	Third Highest	Cement Numbers
16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32				
183	201	104	187	148	212	222	192	245	168	151	187	137	250	201	202	167	280	250	245	1, 29, 24
70	17	74	89	33	168	[c]39	[c]24	12	88	37	[c]50	[d]25+	32+	[c]18	108	89	186[?]	168	133	12, 21, 3
0+	3+	5	3+	0	3+	0	0	0	3+	0	3	0	10[?]	0+	3+	3+	27	17	16	4, 7, 3
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	10	0+	0+	4, 1, 3
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	11	5	5	4, 1, 8
0+	3	0+	3−	0+	0+	0+	0+	0+	8	3−	2	0+	0+	2	9	3	9	8	5	31, 25, 15
173	158	102	192	147	213	229	203	263	209	139	156	127	265	169	220	179	300	265	263	1, 29, 24
71	102	73	[c]61	65	[c]46	[c]44	89	[c]59	35	77	97	[c]67	33	99	68	80	164[?]	141	135	12, 9, 3
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	23	12	11	2, 1, 6
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	3−	0+	0+	2, 3, 8
22	28+	30	11	4	0+	14+	6	3	0	5	9	0	0	3	18	10	30	28+	22	18, 17, 15
166	205	118	199	164	216	192	182	236	177	139	179	113	244	178	215	169	248	244	236	1, 29, 24
[d]3+	[d]3+	[d]7	32	105	33	70	[c]8	58	70	45	42	[d]5	55	[c]18	58	[?]	221	154	142	1, 6, 8
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	59	42	41	1, 2, 6
304	293	248	325	218	353	385	233	407	247	270	270	198	337	268	268	257	407	392	385	24, 1, 22
3+	[d]5	67	0+	3+	[d]0+	[d]3+	22	49	[c]17	34	37	101	8±	40	[c]51	14	133	114[?]	113	13, 12, 8
0	0+	8	0	0+	0	0	0	3−	0	11	3+	12	8[?]	3−	3+	13	22	16	14	13, 10, 6
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	3−	0+	0+	13, 4, 5
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	3+	3	3	5, 2, 8
3−	0+	4	0+	0+	0+	0	0+	6	0+	5	2	0+	0+	4	7	10	10	7	6	32, 31, 24
255	342	241	338	219	347	433	252	465	248	262	323	220	357	306	292	220	465	433	376	24, 22, 1
54	91	[c]54	89	82	..	[d]6±	92	33	72	[c]48	[d]37	89	..	88	74	[c]52	99	93	92	3, 10, 23
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	24	24	18	6, 10, 9
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	0+	..	..	10
41	3+	18	9	3−	0	0	18	7	21+	0	0	13	0	12	24+	3−	41	24+	21+	16, 31, 25
259	348	271	333	201	331	423	267	466	228	284	318	227	360	313	293	254	466	423	387	24, 22, 1
[c]23	17	30	[c]18	77	[c]0+	..	73	18	73	58	38	[d]3±	43	23	59	17	163	78	77	13, 4, 20
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	37	30	28	13, 8, 5
456	516	423	477	411	565	481	486	534	401	604	429	312	427	494	392	356	624	604	588	1, 26, 15
[c]16	15	41	35	[d]3+	15	3−	56	[c]12	[c]0+	5	..	[c]3+	3+	5	[c]0+	[d]3−	56	41	35	23, 18, 19
0	3+	12	0+	0+	0+	0+	20	0+	0+	0+	0	0+	0+	0+	0+	0	20	12	3+	23, 18, 4
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	0+	0+	0+	3, 4, 5
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	3+	3	3	2, 4, 12
3−	3	0+	0+	0+	3−	5	3	3	0+	3−	0	0+	0	3	0+	3	5	3	3	22, 17, 23
496	488	448	510	362	552	515	531	537	411	607	422	335	444	526	420	393	607	605	571	26, 7, 5
22	12	54	19	52	35	25	27	3+	13	..	[d]3±	57	0+	[c]3−	[c]3−	53	57	54	53	28, 18, 32
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	13	12	10	5, 10, 8
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	....
3+	3	21	3	31	3−	3−	10	5	5	0	0	23	0	0	3+	47	47	31	23	32, 20, 28
438	523	413	512	387	579	435	462	457	465	597	427	323	510	478	410	357	613	597	595	5, 26, 3
[c]13+	[c]13	18	[c]3+	[c]0+	[c]18	3+	[c]34	9	0+	..	7	[c]20	3+	..	..	42	54	42	34	14, 32, 23
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	33	9	6	2, 8, 5
496	520	575	626	492	532	555	590	518	512	573	515	537	553	546	462	462	678	630	628	7, 11, 5
13	23	16	7	3−	19	13	16	11	0+	13	7	22	24	3−	31	0+	31	24	23	31, 29, 17
22	26	20	3+	0	4	13	17	3−	0	12	3+	12	21	10[?]	27	12	48	27	27	2, 6, 14
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	13	4	3−	6, 12, 13
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	3−	3−	0+	6, 14, 1
3−	3−	13	0+	0+	3−	6	5	4	0+	3	5	0+	0	0	23	0+	23	13	6	31, 18, 22
536	574	..	637	480	608	561	593	541	517	598	619	608	623	615	498	483	707	688	658	11, 9, 15
18	0+	..	9	23	..	13	11	11	18	..	8	..	12	0	14	6	26	24	23	14, 6, 10
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	61	39	38	6, 1, 2
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	13	7	3−	6, 2, 12
15	0+	..	3+	32	0	26	15	8	18	3	11	0	0	0	22	40	55	40	32	18, 32, 20
557	678	..	637	493	602	630	620	582	480	531	582	542	535	528	478	520	687	678	672	9, 17, 18
0+	3−	..	11	13	..	12	11	9	18	8	..	[c]0+	[c]3+	0+	22	25	33	27	25	14, 4, 32
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	63	62	52	14, 4, 2
71	102	74	89	105	168	70	92	59	88	77	97	101	55	99	108	89	[b]221	[b]168	[b]142	1, 21, 8
70	91	73	89	82	46	44	89	58	73	58	50	89	43	88	74	80	186[?]	154	135	12, 6, 3
54	28+	67	61	77	35	39	73	49	72	48	42	67	33	40	68	74	164[?]	141	133	12, 9, 3
8	8	2	2	13	2	13	22	8	2	8	8	16	13	8	2	2	13	2	13	....
2	22	8	22	22	8	8	8	13	27	27	2	22	27	22	22	8	2	13	8	....
22	11	16	8	27	36	2	27	16	22	22	13	8	8	16	8	16	8	8	2	....

crack, very poor contact.

er the first retest at 3.5 months.

TABLE III.—ORIGINAL AND RETEST TENSILE STRE

1:3 by weight at normal consistency. Water content, 40 to 44 per cent of cement, by weight. All specimens made and
See *Proceedings*, Am. Soc. Testing Mats.

	Specimen Number	Total Age	Time Elapsed Since Last Break	Break Number	Retest Number	Cement															
						1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1	1	3 days......	....	1	..	274	189	167	228	144	218	235	164	160	167	156	143	213	148	187	180
2		3.5 months	3.4 months	2	1	33	57	48	[c]35	50	33	0+	27	11	..	23	114	..	44	..	..
3		6.7 months	3.2 months	3	2	0	0	9	0+	3−	0	0	0+	0	..	3−	0	..	3+	..	..
4		9.0 months	2.3 months	4	3	0+	0	0+	0+	0+	0+	0	0+	0+	0	0+	0+	0	0+	..	..
5		15.0 months	6.0 months	5	4	0	0	0+	0+	8	7	..	3+	3	..	0+	8	..	12	..	..
6		15.0 months	8.3 months	4	3	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	[d]0+
7	2	3 days......	....	1	..	287	179	177	222	137	213	223	173	156	169	165	177	189	152	197	183
8		3.5 months	3.4 months	2	1	23	..	[d]0+	..	..	57[?]	7	13	3+	23	..	63	60	..	..	26
9		9.0 months	5.5 months	3	2	4−	0	0	0	0	0	0+	0	0	0+	0	0	0+	0	..	..
10		15.0 months	6.0 months	4	3	4	..	0	..	..	0	0+	..	..	3	..	0	..	..	..	..
11		15.0 months	11.5 months	3	2	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	0+
12	3	3 days......	....	1	..	277	209	187	253	[a]108	206	215	175	143	153	156	198	238	167	173	165
13		3.5 months	3.4 months	2	1	26	13	3+	..	33	..	[c]0+	22	..	..	..	55	41	..	..	[d]0+
14		15.0 months	11.5 months	3	2	3+	5	0	..	0	..	..	0	..	..	..	6	15	..	..	..
15	4	7 days......	....	1	..	324	344	247	209	318	273	301	229	232	267	264	294	258	294	261	235
16		3.5 months	3.3 months	2	1	21	29	15	..	21	46	53	38	..	..	8	19	..	53	34	..
17		6.7 months	3.2 months	3	2	0+	3+	3+	0	0	3−	3−	38[?]	..	..	0	0	..	3+	0+	..
18		9.0 months	2.3 months	4	3	0+	0+	0	0	0+	0	0	0+	0	0	0	0	0	0+	..	..
19		15.0 months	6.0 months	5	4	0+	0+	0	0	8	3+	0+	4	..	..	0+	4	0+	3+	..	..
20		15.0 months	8.3 months	4	3	..	..	..	..	..	..	..	..	..	..	..	..	..	..	3−	..
21	5	7 days......	....	1	..	337	355	249	192	291	312	312	244	280	242	263	303	326	275	293	252
22		3.5 months	3.3 months	2	1	27	0+	..	..	3+	23	7	27	34	..	8	10	3	34	5	23
23		9.0 months	5.5 months	3	2	0+	0+	0	0+	0+	3−	0+	0+	0	0+	4	3−	0+	3−	..	..
24		15.0 months	6.0 months	4	3	..	0	..	0	0	0	0	0	0	0+	3−	3	..	0+	..	..
25		15.0 months	11.5 months	3	2	..	..	..	..	..	..	..	..	..	..	..	..	..	..	4	0+
26	6	7 days......	....	1	..	312	338	258	212	338	294	289	262	266	252	282	323	292	287	319	240
27		3.5 months	3.3 months	2	1	18	0+	25	..	3+	23	17	..	19	3−	4	..	3+	22	..	0+
28		15.0 months	11.5 months	3	2	8	0	18	..	0+	5	4	0+	8	6	0+	..	0	8	..	..
29	7	28 days.....	....	1	..	463	510	353	347	373	446	446	388	353	375	390	377	454	452	372	355
30		3.5 months	2.5 months	2	1	3−	8	3+	[d]0+	4	0+	0	0+	8	—	3+	3−	..	..	..	8
31		6.7 months	3.2 months	3	2	0	0+	0	0	0	0	3−	0	0	0	0+	0	..	0	..	0+
32		9.0 months	2.3 months	4	3	0	0	0+	0	0+	0	0	0+	0	0	0+	0+	0	0	..	..
33		15.0 months	6.0 months	5	4	..	..	0+	..	0+	3−	0+	3	0+	..	4	0+	..	0	..	..
34		15.0 months	8.3 months	4	3	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	0
35	8	28 days.....	....	1	..	483	519	381	383	425	424	457	416	422	392	397	403	457	454	347	395
36		3.5 months	2.5 months	2	1	[d]0+	20	[d]0+	..	0+	13	3−	3−	6	0+	4	3+	12	4	6	3+
37		9.0 months	5.5 months	3	2	0	0+	0	0	0+	6	0+	4	3−	0	0	0+	0	0+	..	..
38		15.0 months	6.0 months	4	3	0	0+	0	0	0+	0+	0+	..	0+	0	..	0	0	0+	..	..
39		15.0 months	11.5 months	3	2	..	..	..	..	..	..	..	..	..	..	..	..	..	..	0+	0
40	9	28 days.....	....	1	..	491	527	357	358	408	355	482	432	408	422	401	453	464	403	383	414
41		3.5 months	2.5 months	2	1	[d]0+	3+	5	8	..	13	3−	3+	[c]0+	..	..	27	9	..	..	..
42		15.0 months	11.5 months	3	2	..	3+	0	0+	0+	3	0+	0	0+	0	0	6	6	0+	..	..
43	10	3 months...	....	1	..	477	511	492	417	368	389	415	428	438	429	422	453	542	415	408	462
44		3.5 months	0.5 months	2	1	0+	0+	0	0+	0	0+	0+	0+	0	0	0+	0	0	0	0	3−
45		6.7 months	3.2 months	3	2	0	0	0	0	0	0	0	4	3−	0	0+	0	0+	0+	0	0+
46		9.0 months	2.3 months	4	3	0+	0	0	0	0	0	0	0	0	0	0	0	0	0	..	..
47		15.0 months	6.0 months	5	4	0+	0	0	0+	0+	3−	0	3+	0	..	0+	0+	0+	0+	..	..
48		15.0 months	8.3 months	4	3	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	0
49	11	3.0 months	....	1	..	478	531	460	425	438	438	418	498	470	496	458	474	533	482	476	461
50		3.5 months	0.5 months	2	1	0+	0+	0+	0	0+	0+	3−	0+	0	0+	0+	3−	0+	0+	0	0
51		9.0 months	5.5 months	3	2	0	0	0	0	0	0	3−	0	0	0	3+	0+	3−	0	..	..
52		15.0 months	6.0 months	4	3	0	0	0	..	0	0	0+	..	0	0+	..	0+	0	0	..	..
53		15.0 months	11.5 months	3	2	..	..	..	..	..	..	..	..	..	..	..	..	..	..	3−	0+
54	12	3.0 months	....	1	..	489	557	407	485	404	443	488	417	427	475	483	514	508	476	432	432
55		3.5 months	0.5 months	2	1	0+	0+	0	0+	0	0+	0	0	0	0+	0	..	0+	0+	3−	0
56		15.0 months	11.5 months	3	2	0+	0	0	10	6	4	0	3+	..	4	3	0+	0	..	..	..
57	Highest Retest..............................					33	57	48	35	50	57[?]	53	38	34	23	23	114	60	53	34	26
58	Second Highest Retest.....................					27	29	25	10	33	46	17	38[?]	19	6	8	63	41	44	6	23
59	Third Highest Retest.......................					26	20	15	8	21	33	7	27	11	4	8	55	15	34	5	8
60	Highest (Line Number)................					2	2	2	2	2	8	16	16	22	8	2	2	8	16	16	8
61	Second Highest (Line Number)........					22	16	27	56	13	16	27	17	27	28	16	8	13	2	36	22
62	Third Highest (Line Number).........					13	36	16	41	16	2	8	2	2	56	22	13	14	22	22	30

[a] Strength somewhat erratic. [b] Highest, second, and third apply to vertical columns only. [c] Crack, broken halves not in full contact. [d] Wide crack,

NOTE.—(0+) indicates slight adhesion up to about three pounds.

Note that Specimens 1-3, 4-6, 7-9, and 10-12, respectively, are companion groups. Members of each group had identical treatment until after first retest

PLATE III.
PROC. AM. SOC. TESTING MATS.
VOL. 29, PART II.

GILKEY ON AUTOGENOUS HEALING OF CONCRETE.

STRENGTH VALUES OF STANDARD SAND BRIQUETS.

made and originally tested as part of the 1927 cooperative test program under the Society's Committee C-1 on Cement.
ng Mats., Vol. 28, Part I, p. 261 (1928).

Cements 1–32																	Highest	Second Highest	Third Highest	Cement Numbers
16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32				
180	168	177	197	113	269	237	212	273	206	164	224	142	268	177	202	195	274	273	269	1, 24, 21
..	41	37	[d]0+	..	[d]0+	48	19	0+	..	..	..	20+	..	..	..	..	114	57	50	12, 2, 5
..	3−	0+	0+	0	0	0	0+	0	0	0	0	3+	..	0	0	0	9	3+	3+	3, 14, 28
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	0+	0+	0+	1, 3, 4
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	12	8	8	14, 5, 12
[d]0+	10	3−	0+	..	0+	6	0	0+	..	..	[c]0+	11	..	0+	..	..	11	10	6	23, 17, 22
183	170	154	198	138	267	242	205	265	211	158	219	138	246	217	213	177	287	267	265	1, 21, 24
26	..	..	[d]5	..	[d]3−	76	..	..	17	[d]3−	[d]0+	3−	..	4	65	67	76	67	65	22, 32, 31
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	4	0+	0+	1, 7, 10
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	4	3	0+	1, 10, 7
0+	..	..	0+	..	0+	11	..	..	0+	..	0	5	..	0+	16	12	16	12	11	31, 32, 22
165	177	157	203	146	222	247	209	298	217	163	[a]183	166	279	184	220	191	298	279	277	24, 29, 1
[d]0+	[d]0+	[d]0+	46	..	..	143	..	..	0	..	..	..	12	..	3+	..	143	55	46	22, 12, 19
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	15	6	5	13, 12, 2
235	284	279	299	212	322	340	310	378	308	261	278	212	331	238	288	278	378	344	340	24, 2, 22
..	..	[d]3−	26	..	[d]0+	33	..	..	5	[d]0+	..	..	13	16	..	..	53	53	46	7, 14, 6
..	..	0	0	..	0	3−	0	0	3+	0+	0	..	3	0+	0	0	38[?]	3+	3+	8, 2, 3
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	0+	0+	0+	1, 2, 5
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	8	4	4	5, 8, 12
..	..	..	0+	..	0	4	0	0+	4	..	..	..	4	0+	0+	..	4	4	4	22, 25, 29
252	293	288	291	212	362	304	289	388	268	227	301	189	313	240	256	283	388	362	355	24, 21, 2
23	..	18	38	..	..	13	..	16	4	[c]18	[d]0+	19	0+	..	..	..	38	34	34	19, 9, 14
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	4	3−	3−	11, 6, 12
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	3	3−	0+	12, 11, 10
0+	0+	3	3−	..	..	12	..	7	0+	10	..	3	0	0	0+	..	12	10	7	22, 26, 24
240	298	295	312	192	317	334	302	359	257	242	264	228	323	270	284	258	359	338	338	24, 2, 5
0+	..	38	32	..	..	19	17	12	..	..	[d]0+	..	18	0+	..	..	38	32	25	18, 19, 3
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	18	8	8	3, 1, 9
355	368	383	439	382	387	422	368	345	391	414	400	328	462	387	317	356	510	463	462	2, 1, 29
8	0+	3+	6	0+	4	0+	..	0+	..	..	0+	..	0+	0+	..	11	11	8	8	32, 2, 9
0+	0	0	0	0	0+	0	0	0	0	0	0	0	0	0	0	0+	3−	0+	0+	7, 2, 11
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	0+	0+	0+	3, 5, 8
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	4	3	3−	11, 8, 6
0	..	..	3−	0+	3−	0+	0	0+	..	..	0	0	0+	0+	..	0	3−	3−	0+	19, 21, 20
395	377	427	371	341	425	432	364	395	401	412	414	330	409	368	387	393	519	483	457	2, 1, 7
3+	..	0	..	11	0+	0+	..	0+	0+	8	[d]0+	..	10	0+	..	0+	20	13	12	2, 6, 13
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	6	4	3−	6, 8, 9
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	0+	0+	0+	2, 5, 6
0	0	0	..	0+	0+	..	..	0+	0	8	0	..	0	0+	0	0+	8	0+	0+	26, 15, 20
414	362	410	418	315	361	466	330	434	392	413	408	326	457	376	354	357	527	491	482	2, 1, 7
..	0	..	..	3+	5	0+	..	3+	0+	3−	0+	..	[c]0+	0	[d]0+	0	27	13	9	12, 6, 13
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	6	6	3+	12, 13, 2
462	445	402	467	500	483	628	461	438	433	431	409	483	498	418	400	432	542	528	511	13, 22, 2
3−	0	0	0	..	0	0+	0	0+	0+	0+	0+	..	0+	0+	0+	3±	3±	3−	0+	32, 16, 1
0+	0+	0	0	0	0+	0+	0	0+	0	0	0	0	0	0	3−	0+	4	3−	3−	8, 9, 31
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	0+	..	..	1
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	3+	3−	0+	8, 6, 1
0	0+	..	0	..	0	3	0	0+	3−	0+	0+	0+	0	0	0+	0+	3	3−	0+	22, 25, 17
461	441	422	478	484	434	498	397	448	525	452	478	482	456	432	445	479	533	531	525	13, 2, 25
0	0	0+	0	0+	0+	0	0+	0+	0+	0+	0+	0+	0+	0+	0	0	3−	3−	0+	7, 12, 1
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	3+	3−	3−	11, 7, 13
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	0+	0+	0+	7, 10, 12
0+	0	3	..	3−	3−	0	0+	0+	0+	0	0	0+	0	4	0+	0	4	3	3−	30, 18, 15
432	413	427	457	488	491	458	340	501	441	493	450	420	481	412	413	458	557	514	508	2, 12, 13
0	0	0+	0	0+	0	0	0+	3−	0	3±	4	0	0+	0+	3−	0+	4	3±	3−	27, 26, 15
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	10	6	4	4, 5, 6
26	41	38	46	11	5	143	19	16	17	18	4	20+	18	16	65	67	[b]143	[b]67	[b]65	22, 32, 31
23	3−	37	38	3+	4	76	17	12	5	10	0+	19	13	4	16	12	114	57	50	12, 2, 5
8	0+	18	32	3−	3−	48	0+	3+	4	8	0+	11	12	4	3+	11	76	55	46	22, 12, 19
8	2	27	13	36	41	13	2	22	8	22	55	2	27	16	8	8	13	8	8	....
22	3	2	22	41	30	8	27	27	16	25	6	22	16	8	11	11	2	2	2	....
30	13	22	27	53	8	2	3	41	20	36	8	6	13	53	13	30	8	13	13	....

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e crack, very poor contact.

rst retest at 3.5 months.

draw conclusions one finds two factors in either or both of which the explanation may lie:

1. The younger the specimen at initial test the greater the expected healing. Some of the earlier test data of other series do not bear this out. A few 7-day initial breaks developed less retest strength than those initially tested at 28 days. In spite of this lack of complete accordance it seems that this explanation is a reasonable one and must account for some of the extra strength recovery of the 3 and 7-day specimens as compared with the 28-day and 3-months initial tests in Tables II and III.

2. It also seems reasonable to assume that the time available for curing should be an important factor. It is apparent that the older specimens were greatly handicapped on the matter of time, since the 3-day specimens had nearly 3.5 months for recovery while the 3-months specimens had but two weeks. On the other hand, an extra three or six months between the second and third retest (third and fourth breaking) does not appear to add much to the extent of recovery, as can be noted by comparing lines 3, 9, etc.

These are but a few of the distinctive items on which Tables II and III can be made to shed independent light. Sometimes the light answers questions. More often than not it raises others.

EXPLANATION OF THE HEALING ACTION

It seems evident that the main healing action is largely a resumption or continuation of the regular setting or hardening processes. It is therefore evident that healing will occur under essentially the same conditions as normal curing, namely, in the presence of moisture at ordinary temperatures.[1]

The delayed chemical processes probably arise from two causes:

1. Cement under favorable curing conditions is active (that is, the curing processes are not completed) for at least ten years, and probably much longer. These uncompleted reactions can occur across a crack and adhesions result. The fact that no calcium aluminates were found is in line with this deduction, since the aluminates complete their activity or function in the setting process at a very early age (usually within 24 hours).

2. The coarser particles of the cement are doubtless much slower to hydrate than are the finer ones, and these coarse particles conceivably form a reserve supply of unhydrated material that is available for the repair of damage, or that will add to the later strength. This

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[1] In his discussion of this paper Mr. R. H. Bogue supplies the technical explanation of the actions probably involved.

deduction is quite in line with the recognized fact that finer grinding accelerates the rate of strength gain. It would also seem to indicate that the ultimate strength should not be greatly influenced by fine grinding unless a more favorable dispersion or distribution of cement particles was attained thereby, or unless the favorable curing environment was terminated before the coarse particles had responded.

If the foregoing deductions are correct, it would appear that analyses made on specimens initially broken and later retested at different ages should show different proportions of the respective healing constituents. Early breaks might even show some alumina present, from the coarser cement particles, at least.

Tables II and III supply data on the healing performance of 32 brands of cement. These cements have been thoroughly analyzed. Might it not be possible for some well-equipped laboratory to compare these results with the chemical and mechanical analyses of the different cements to see if there appears to be any correlation between healing and chemical composition or between healing and fineness? It is true that the "crack, healed area, and eccentricity variables" are still present as disturbing factors. Moreover it is apparent that the ordinary No. 200 sieve analysis is probably a poor criterion for judging fineness in its relation to rate of set or healing. The active particles are probably much finer than those that will just pass the No. 200 sieve, so that one cement might all pass and be relatively coarse while another might have a considerable residue on the sieve and still be a much finer cement except for the residue. Thus, if these 32 brands of cement were not investigated with the air analyzer or other more discriminating device than the No. 200 sieve, it might not be possible to obtain any comparison of value. Among the 32 cements of the Committee C-1 investigation[1] there was great divergence in the rate of strength gain with age. If adequate fineness data are available, in addition to the chemical analyses, which were made, it appears possible that much of a very basic nature might be brought out by a search conducted along these lines.

COMMENTS AND POSSIBLE APPLICATIONS

In no case did tensile healing attain in magnitude the full strength at initial test. If compared with the strength that an unbroken specimen would have attained at the age of retest (which is, after all, the logical comparison) it must be recognized that direct tensile healing will in all cases be rather small. Thus a healed tensile crack can never be expected to resist as much stress as that which caused it. In spite of this fact there are many instances in which tensile autogenous healing can be expected to be of practical value. Suppose, for example, that concrete piling develops cracks in the process of handling or driving. Once in place, the cracks are tightly closed and the sort of stress that damaged the piles is no longer present. Moisture is available for curing. The cracks may be expected to heal, the reinforcement will be properly protected and the resultant damage is likely to be nil. On the other hand, cracks that occurred in a building exposed to dry air could not be expected to heal. In the case of pavements, dams, and many other types of structure, the autogenous healing will often serve as a reserve strength factor not heretofore recognized.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, Fig. 10, p. 296 (1928).

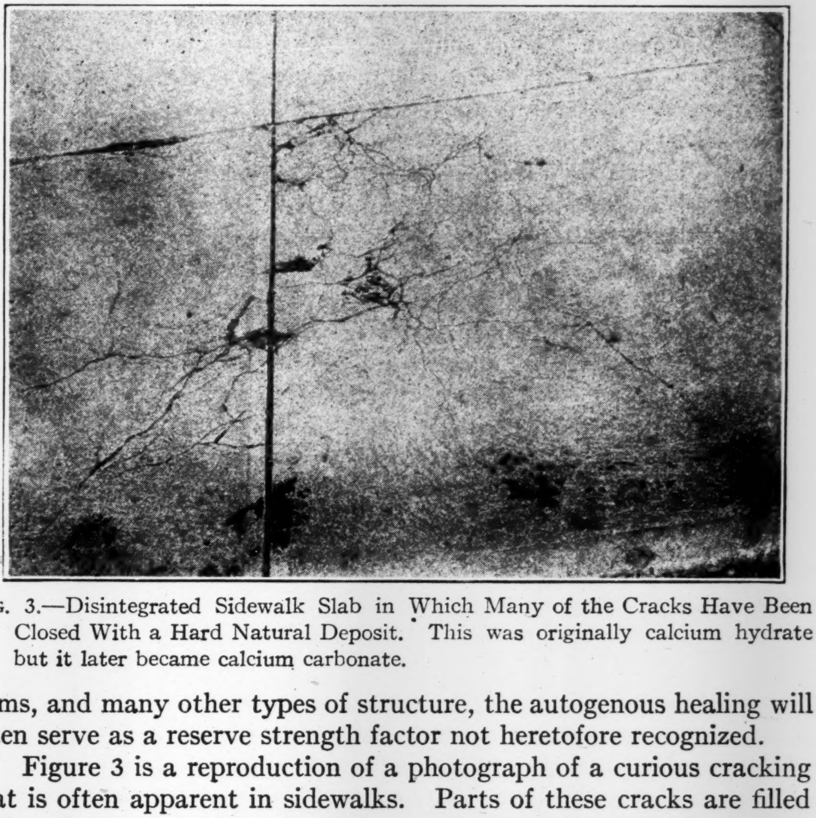

FIG. 3.—Disintegrated Sidewalk Slab in Which Many of the Cracks Have Been Closed With a Hard Natural Deposit. This was originally calcium hydrate but it later became calcium carbonate.

Figure 3 is a reproduction of a photograph of a curious cracking that is often apparent in sidewalks. Parts of these cracks are filled with a dark-colored hard bead that seals it entirely. Evidently this hard bead is a growth that has occurred after the crack formed.

Analyses of the filling material show it to be calcium carbonate diluted with some dirt. Evidently calcium hydrate was brought from within the concrete by moisture. The carbon dioxide of the air combines to form the carbonate. Mr. R. D. George[1] made the analysis. His results agree with those of the Portland Cement Association. The beads are very shallow. The action is evidently not an autogenous healing of the sort previously described but is rather a redeposition.

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[1] Head, Department of Geology, University of Colorado, Boulder, Colo.

That the contents of this paper be of direct practical application is probably not so essential as that it should contribute something that is new to current concrete knowledge. It is hoped that, in some slight degree, this may have been done.

SUMMARY AND CONCLUSIONS

1. Tensile autogenous healing occurs for the same general curing conditions as does initial strength gain, that is, in the presence of moisture and moderate temperature. It does not occur in a dry specimen.

2. It occurs for either standard Ottawa or natural sands and probably for all portland cements.

3. Mixtures as lean as 1:5 parts standard sand by weight showed healing to as great a percentage of initial strength as did the richer mixtures. This was the leanest mixture tried.

4. Healing occurred for mixtures varying from the very dry to the very wet (water-cement ratios from 0.20 to 1.20 for sand-cement mortars).

5. Healing occurred for all ages of initial breakage from 3 days to 2¼ years.

6. As many as six tests (five retests) have been made on many specimens and measurable strengths were found for some at the last test. This paper records no results beyond 5 tests (4 retests).

7. The strength, in general, is less for each succeeding test, and the initial strength is rarely approached in tensile healing tests.

8. Specimens left in dry air for months are inert but healing will resume upon re-immersion.

9. Distinct crystallin structure is visible in many of the retest breaks.

10. Healing seems to extend from the center outward and never reaches nearer than about ⅛ in. from the outer edge. It does not appear to be a re-deposition of soluble salts across the cracks, as has been occasionally suggested as the reason for compressive healing.

11. Whether the water is running or still, and carrying many curing specimens for the volume of the water, seems to have no major qualitative effect upon the healing. (Tentative.)

12. Rich mixtures heal more than lean ones, but the average percentage healing in terms of original strength is not far different

for the two of them. It is slightly higher for the lean mixture. (See also No. 3.)

13. The strongest specimen at initial test does not necessarily prove to be the one giving the greatest retest strength.

14. The probable explanation of healing is that it is a continuation of processes not yet completed because of (*a*) natural slowness of the calcium silicate to fully react, (*b*) slowness of hydration of large particles that are in the vicinity of the fracture.

15. Several have noticed and mentioned compressive, flexural and bond healings. This is probably the first experimentation and record of tensile healing. All are, of course, but different manifestations of the one healing phenomenon.

16. The evidence set forth may have little of a practical nature just now, but it is felt that it may be found of value in connection with the general study of cement and the setting process. Moreover, while not of apparent major importance, a knowledge of autogenous healing and the conditions under which it may be expected to occur has a number of possible applications. It may prove important in connection with the remedying of overstrain and hair cracking. Small cracks from driving concrete piling, for example, could be expected to heal in moist soil sufficiently to protect the reinforcement and not impair usefulness from the standpoints of strength and durability. Some sidewalk cracking illustrates redeposition rather than healing.

17. While the general conditions, under which healing will or will not occur, have been found with fair certainty, no quantitative healing law has been discovered.

18. Because of the possible relationship between fineness of grinding and chemical variations in the composition of the cement, to both the healing process and the rate of normal strength gain with age, it is urged that the data of Tables II and III be studied in connection with the composition and fineness of the 32 cements involved. If that be not practicable, the same phases could be studied by one of the larger laboratories, a new program being launched for the purpose.

Acknowledgments:

Russell D. George, Head of the Department of Geology, has greatly aided in the work by virtue of his advice, interest and criticism. Mr. R. D. Crawford, also of the Department of Geology, aided in an earlier attempt at identification of crystal structure. William Eager, Cleland Feast, Smith Ketchum, and Sartwell Egerton, students or former students in Civil Engineering, assisted in various phases of the work. Mr. W. E. Brockway, Supervising Engineer, has also cooperated.

DISCUSSION

Mr. R. H. Bogue[1] (*presented in written form*).—It is known that the compounds that make for great strength in portland cement are tricalcium silicate and dicalcium silicate.[2] Of these, the former reacts with water at such a rate that, with ordinary mixes and moist curing, it is completely hydrated between 3 and 12 months. The dicalcium silicate reacts more slowly, becoming conspicuous only after about a month and continuing for some years. Changes in fineness or curing may greatly alter the rate of the reactions.

Thus set cement usually will contain a quantity of hydraulic material which is capable of acting as a bonding agent when brought in contact with water. It is immaterial whether the water is brought to it by placing the specimen in water; or by grinding the specimen and regaging with water; or by breaking the specimen, tying together, and again placing in water. The first process is ordinary or extended curing; the second is retempering of set cement; the third, autogenous healing. But they each are consequences of the same action—the continuation of the normal hydration which brings about the hardening of cement. The principal products of this hydration are crystalline calcium hydroxide, tricalcium aluminate hydrate, and calcium sulfoaluminate and amorphous hydrated calcium silicate and an iron-containing phase not definitely identified. Other materials in small amounts may be present. The calcium hydroxide may become carbonated where exposed to the air or carbonated waters.

There are several physical forces which may be expected to aid materially in the healing of two parts of a specimen which are bound together. These have been discussed by Donnan.[3] (1) The solubility of the solid phase will be increased due to compressive stress. The material thus dissolved will be precipitated in contact with unstressed portions, cementing the granules together by the formation of "bridges" of precipitated material. (2) The greater solubility of the very fine granules in contact with a solution that is saturated with respect to the coarser granules permits the former to be redissolved and reprecipitated, again forming "bridges" of bonding material.

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[1] Research Director, Portland Cement Fellowship, U. S. Bureau of Standards, Washington, D. C.

[2] P. H. Bates and A. A. Klein, U. S. Bureau of Standards *Technologic Paper No. 78* (1917).

[3] F. G. Donnan, *Transactions*, Faraday Soc., Vol. 14, p. 12 (1919).

(3) The metastable forms have higher solubility than the stable forms with the consequence that the former are dissolved and reprecipitated into the stable form.

It is concluded that the healing action represents a continuation of the hydration of the hydraulic compounds of the cement, supplemented by physical stresses which aid in the formation of precipitated bonds between the severed grains. The compounds formed at the break may be expected to be identical to those formed during the normal course of hydration.

MR. D. A. ABRAMS.[1]—In 1913 I presented a paper before this Society[2] in which tensile autogenous healing was observed in making a load test of a 40-ft. reinforced concrete highway bridge. A load of 106 tons was applied when the bridge was about 3 months old; this load formed distinct diagonal tension cracks in both ends of the girders. The bridge stood exposed to the weather for 3 years before it was re-loaded. Upon re-loading about 6 times the original load was applied to the girders. In many cases the old cracks never opened, but new cracks were formed 2 to 4 in. from the old cracks. This phenomenon was not then referred to as autogenous healing of concrete; this term was first used in a later investigation of the compressive strength of concrete and bond between concrete and steel.[3]

MR. H. J. GILKEY[4] (*author's closure by letter*).—The author is very grateful for Mr. Bogue's thorough and lucid explanation of the healing action. It is especially gratifying to find that the author's nontechnical explanation of healing as "a continuation or resumption of the normal curing processes" is supported.

As stated on page 594 of the paper, any healing, whether compressive, flexural, shearing or tensile, involves adhesion and is after all but a varied manifestation of the same phenomenon. The respect in which these tests differ from the healing manifestations recorded by Abrams and others is that these are tests of healing of fully severed members. The strength recovery in beams, in which the crack does not extend entirely across the section, is no doubt due in part to direct tensile healing and in addition to changes in the strength of the concrete in the compressive and unsevered portion of the cross-section. As is true for compressive healing, the phenomenon is complicated by overlapping factors.

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[1] Director of Research, International Cement Corp., New York City.

[2] D. A. Abrams, "Test of a 40-foot Reinforced Concrete Highway Bridge," *Proceedings*, Am. Soc. Testing Mats., Vol. XIII, p. 884 (1913).

[3] See *Bulletin 71*, Engineering Experiment Station, University of Illinois, 1913.

[4] Associate Professor of Civil Engineering, University of Colorado, Boulder, Colo.

The author has often alluded to the earlier observations of Abrams relative to healing in flexure, compression and bond. So far as the author knows Abrams was the first to notice and to call attention to the phenomenon and, as Abrams states in his discussion, the term "autogenous healing" was originated by him.

In his earlier paper on the subject, the author attempted to catalogue more definitely something that had received but a passing and secondary notice, first by Abrams and later by others. This first venture into the field of *direct tensile healing of fully severed members* was undertaken with a view to getting at the underlying causes and to obtaining more positive quantitative measurements. With the help of Messrs. George, Crawford and Bogue, it is felt that that purpose has been accomplished.

SUGAR IN SAND: A QUICK TEST TO DETERMINE ITS PRESENCE BACTERIOLOGICALLY

By N. H. Roy[1]

Synopsis

The effects of sugar[2] in portland-cement mortar and concrete have been discussed by many authors, and the serious and costly effects of its presence are so commonly known that a discussion of them is not necessary in this paper. Sugar has been found in sand from some deposits, and doubtless there have been many others in which its presence was unknown and unsuspected. The presence of sugars in natural sands may account for many cases of concrete that will not set or harden or that have low strength. As the effects of even small amounts of sugar generally are to ruin concrete,[3] a method for detecting its presence in sand is discussed in this paper, together with a statement of the classes of sugars found naturally in fields and streams, and of the bacterial action that takes place in nature.

The proposed test is the reversal of a common bacterial test—the use of a common harmless bacterium, *Aerobacter aerogenes* (*Bacterium aerogenes*), to test for the presence of sugar.[4] A washing from the sand is placed in a test tube together with a small quantity of bouillon, the mixture is inoculated with *Aerobacter aerogenes* and set aside in a warm place for a short time. The presence of sugar is revealed by the liberation of carbon dioxide which is easily collected in a fermentation tube to furnish visual quantitative evidence. Carbon dioxide will not be liberated by the bacteria from organic materials such as tannic acid. The method is rapid and reliable and will detect very minute quantities of sugar.

Supporting data and photograph showing the results of tests are included.

Introduction

The desirability of having a method for detecting the presence of sugar[2] in sands arose when several sands submitted for concrete work were found to contain sugar. Investigation of deposits of sand in creeks and rivers revealed in some cases the presence of sugar, and in other cases substances that could very easily and naturally be converted into sugar. Any plant growth that contains starch, such as wheat, oats, rye, beans, and potatoes can easily be converted to maltose and then to simple sugars by bacterial action in fields and streams. Likewise cellulose (leaves, wood fiber, etc.) may be converted to sugar by similar action. Further bacterial action may reduce these sugars to carbon dioxide and alcohol, and finally to water and more carbon dioxide. It is therefore possible for sand deposits to receive sugar contamination from natural causes; and it has happened in a number of cases that sands for concrete were taken from the bank or pit just when the sugar stage of bacterial action had been reached.

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[1] Department of Theoretical and Applied Mechanics, University of Illinois, Urbana, Ill.

[2] The term sugar, as used herein, is applied to fermentable substances.

[3] The Bibliography of a paper by D. A. Abrams on "Effect of Tannic Acid on the Strength of Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. XX, Part I, p. 338 (1920) contains references to many papers dealing with the effects of sugar in mortar and concrete.

[4] Aldo Castellani and F. E. Taylor, "The Mycological Detection and Determination of Certain Carbohydrates and Other Carbon Compounds in Pathological Work," *British Medical Journal*, Vol. 177, J. 2, p. 855, December 29, 1917; and "Further Observations on a Mycological Method to Identify Various Sugars and Other Carbon Compounds," *The Journal*, Am. Medical Assn., February 20, 1926, Vol. 86, p. 523.

Platinum
Wire

FIG. 1.—Platinum Needle Used in Tests.

A series of tests was conducted in the laboratory to find, if possible, a bacterium that would (1) show the presence of sugar if it is present; (2) give negative results for all other substances present, especially organic matter; and (3) give some indication of the amount of sugar present.

MATERIAL AND EQUIPMENT

Materials and equipment used in making tests described herein, were as follows:

(*a*) Test or culture tubes of approximately 25 cc. capacity,

(*b*) Fermentation tubes of the Durham type of approximately 10 cc. capacity,

(*c*) Platinum needle (for inoculation),

(*d*) Five classes of sugar, saccharose, maltose, glucose, levulose, and lactose,

(*e*) Constant temperature room for incubation, and

(*f*) Bacteria and bouillon.

TEST METHODS

In order to duplicate as nearly as possible the actual conditions of testing a sand sample, a washing from a sand was taken, from which solutions were made of each of the five sugars in varying concentrations. Ten cubic centimeters of bouillon were placed in each test tube, the fermentation tube was filled with bouillon, and inverted inside the test tube. Ten cubic centimeters of sugar solution were added to each test tube containing bouillon. The test tubes were arranged so that there were six tubes containing varying amounts of each of the five sugar solutions. Tubes containing washings of sands and distilled water were also included for check purposes. All tubes were then inoculated with *Aerobacter aerogenes* (*Bacterium aerogenes*), stoppered with cotton, and placed in a temperature of 38° C. for 20 hours. The culture of *Aerobacter aerogenes* was kept in a tube of agar (slant) at room temperature (25° C.). To inoculate a tube with *Aerobacter aerogenes*, a platinum needle is taken with a loop at the end (Fig. 1). The loop is passed through a flame, then filled with bacteria from the growth in the culture tube and inserted in the tube of liquid to be tested. The bacteria are removed by moving the needle about in the liquid.

TABLE I.—VOLUME OF GAS LIBERATED FROM VARIOUS SUGAR SOLUTIONS.

Kind of Sugar	Carbon Dioxide, per cent by volume					
	Sugar, 0.75 per cent by weight	Sugar, 0.50 per cent by weight	Sugar, 0.25 per cent by weight	Sugar, 0.05 per cent by weight	Sugar, 0.005 per cent by weight	Sugar, 0.0005 per cent by weight
Saccharose.......	10	6	2	1	0	0
Maltose.........	16	4	1	bubble	bubble	0
Glucose..........	6	2	1	0	0	0
Lactose..........	7	1.5	bubble	0	0	0
Levulose.........	1	1	1	0.5	bubble	0

DATA AND DISCUSSION OF THE DATA

The amount of sugar present in a set of test tubes varied from 0.75 per cent by weight in the first tube to 0.0005 per cent by weight in the sixth tube. There were five sets of the five classes of sugar tested which were as follows: (*a*) saccharose, or cane sugar; (*b*) maltose, or converted starch; (*c*) glucose, or converted maltose; (*d*) levulose, or fruit sugar; and (*e*) lactose, or milk sugar. The likelihood of their occurrence in sands is probably as follows: (*a*) maltose, glucose, and saccharose are quite likely to appear in some form; (*b*) lactose is not likely to appear; and (*c*) levulose quite unlikely.

Carbon dioxide was liberated by the bacterial action and caught in the inverted fermentation tubes. After inoculation it was found that from three to five of the tubes out of each set of six showed the presence of carbon dioxide. Table I gives the percentages of carbon dioxide caught in each of the thirty test tubes. It is of extreme interest to note that there was liberated a comparatively large amount of gas with the highest percentage of sugar and progressively less gas in each succeeding tube as the percentage of sugar became less.

This is well shown for one of the sugars in Fig. 2. In no case was there gas with the lowest percentage of sugar, the percentage having been chosen low enough to prevent the formation of enough gas to be seen. From a visual inspection of the amount of gas caught in the fermentation tube, with practice one may closely estimate the percentage of sugar present, when the kind of sugar is known. The various kinds of sugar give varying percentages of gas. Therefore, when testing a sand for the presence of sugar, if gas in any quantity is obtained it will be known that some class of sugar is present. The strength test of the sand should then be very carefully watched. Washes from four sands in the laboratory and the tube of distilled water (all containing bouillon and the bacteria) gave no gas.

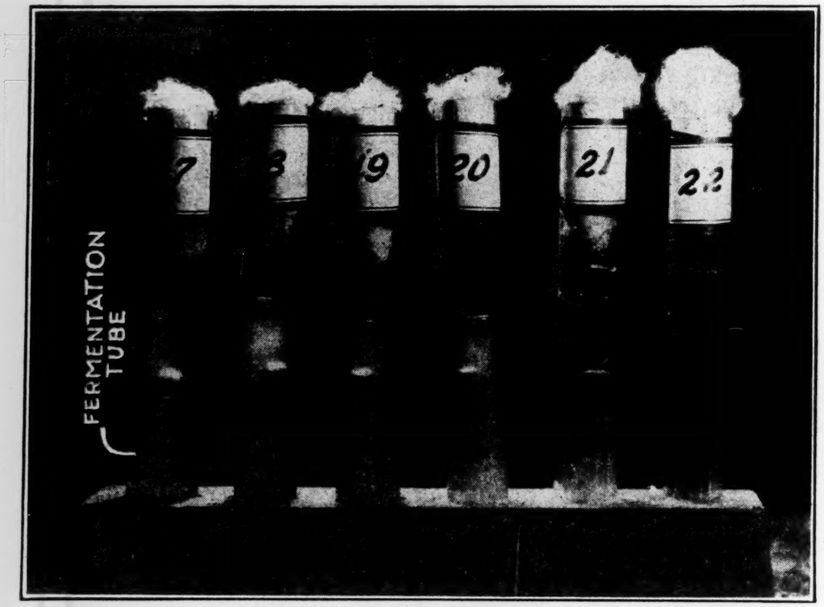

FIG. 2.—Six Percentages of Saccharose Sugar, Showing Amount of Carbon Dioxide Liberated.

This series of tests gives results similar to those obtained by the author in a series of tests at the University of Texas in 1924. Further tests are at present being conducted to give additional information or another possible method of detecting the presence of sugar in sand.

SUGGESTED PROCEDURE

It is realized that probably there have not been enough tests made to cover all conditions: there is much yet to be done. Some points of interest may be (*a*) the effects of time and temperature of incubation on the amount of gas liberated, (*b*) the effects of various mixtures of sugar on gas liberation, and (*c*) the possibility of putting *Aerobacter aerogenes*, the bacterium, into tablet form so as to simplify its use. Results of the above tests have been very satisfactory and consistent. Consequently the method outlined above for the detection of sugar in sand is suggested for further study.

It is not necessary that test tubes, water, or hands be sterile. The only precaution necessary is that there must not be any sugar contamination from water, tubes, etc. However, the culture of *Aerobacter aerogenes* should be kept clean. This may be done by passing the platinum needle through a flame before using, and burning the cotton stopper of the culture tube slightly before replacing after use. Test tubes of bouillon with the inverted fermentation tubes inside may be made up at any time in quantities. The plain bouillon or plain broth may be made by dissolving 5 g. of peptone and 3 g. of beef extract in 1000 cc. of water. It is necessary to heat the water to dissolve the beef. The bouillon may be kept in a sterile flask or sterile tubes indefinitely, preferably in a cool place.

A culture of *Aerobacter aerogenes* may be grown and kept on agar. A tube in which the agar is slanted is convenient for use. The growth may be plainly seen on the surface of the agar. It is well to transfer the loop of this bacteria to the surface of a new tube of agar every two or three weeks so that a virile culture may be on hand at all times. The culture may be kept at room temperature or about 25° C.

Acknowledgments.—Acknowledgment is hereby made to I. McK. Lewis, University of Texas; F. W. Tanner, University of Illinois; and H. R. Thomas, University of Illinois, for their assistance and suggestions.

DISCUSSION

Mr. F. O. Anderegg[1] (*presented in written form*).—In working with a portland cement clinker somewhat more finely ground than usual, we have found that small amounts of sugar produced a flash set. The same phenomenon, although not quite so obvious, occurred when the cement was ground less finely. Such an acceleration of the initial set of portland cement would explain the numerous results recorded in the literature of the disastrous effects caused by the presence of sugar in the raw materials used in making concrete.

Mr. P. J. Freeman[2] (*presented in written form*).—The method suggested by Mr. Roy is a step in the right direction because it affords one more method for obtaining precise information about a material concerning whose properties we are usually but ill informed. I look with favor on methods of testing which tend to eliminate ordinary physical methods of manipulating materials, and it would appear that this method would eliminate the necessity for testing sands for physical strength of mortar when the presence of sugar is suspected.

Sand containing sugar from natural sources usually contains other vegetable residues more easily determined, but cases of artificial contamination during loading and shipping are not uncommon. This method is hardly simple enough for routine use in the hands of ordinary laboratory operators who are unfamiliar with bacteriology. The laboratory helper would probably bungle it more often than not, but for laboratories having a competent chemist familiar with bacteriological technique, the method undoubtedly has great possibilities.

Water for mixing concrete is also occasionally contaminated with vegetable residues, and a modification of this method might be used for the detection of sugar therein, and therefore afford an additional test to the unsatisfactory physical test now employed for determining the suitability of water for use in concrete.

Mr. J. C. Witt[3] (*by letter*).—There is no question that sugar is harmful to concrete, even when present in very small quantities, regardless of its source. For that reason any satisfactory test to reveal its presence in any of the materials used is desirable and of value. Mr.

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[1] Senior Industrial Fellow, Atlas Portland Cement Co. Industrial Fellowship, Mellon Institute of Industrial Research, Pittsburgh, Pa.

[2] Chief Engineer, Bureau of Tests and Specifications, Department of Public Works, Allegheny County, Pittsburgh, Pa.

[3] Chemical Engineer, Chicago, Ill.

Roy's test is of special interest in that it apparently adds another science to those already employed in the manufacture and use of cement. This is the first instance of the application of bacteriology to concrete making that has come to my notice.

Probably the use of the test will be limited because in many laboratories where concrete materials are tested, bacteria cultures and the necessary equipment and technique are not available.

MR. N. H. ROY[1] (*author's closure by letter*).—Further tests are now being made with the expectation of simplifying the procedure, especially with regard to the form in which the bacterial culture may become available. At present, it seems probable that the culture needed for the test can be put into a dry form which will greatly simplify the test.

It is the author's belief that, with the aid of such discussions as have been presented by Messrs. Anderegg, Freeman, and Witt, a quick workable test for the detection of sugar in sand or water may be evolved.

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[1] Department of Theoretical and Applied Mechanics, University of Illinois, Urbana, Ill.

TESTS OF POWDERED ADMIXTURES IN CONCRETE

By Duff A. Abrams[1]

Synopsis

Two series of concrete tests (266 batches) were carried out in the Research Laboratory of the International Cement Corporation, New York City, in a study of effect of powdered admixtures on strength and workability. Seven commercial and 7 non-commercial admixtures were used. In general, 3 percentages were used, ranging from 2 to 15 per cent of weight of cement; in certain instances this replacement was continued to 50 per cent.

A study was made of the influence of quantity of cement and water on the strength and workability of concrete without admixtures. Diagrams show the inter-relations of these factors and indicate how they are influenced by admixtures. The flow test was found to be a satisfactory measure of workability.

The admixtures which were slightly hydraulic showed small increase in strength. The non-hydraulic admixtures showed reduction in strength up to about 5 per cent for each 1 per cent admixture in terms of weight of cement. Colloidal clays and diatomaceous earth gave the greatest reduction in strength. In general, the reduction in concrete strength was greater at 1, 2 and 3 days than at 7 days to 3 months.

None of the admixtures improved the workability as measured by the flow test.

There was an intimate relation between the normal consistency of the non-hydraulic admixtures and the reduction in concrete strength; a similar relation was found for the "water factor" of these admixtures.

Introduction

There is a widespread interest at the present time in powdered admixtures in concrete. The engineer and contractor are invited to use them because of alleged increased workability, waterproofing, increased yield and for other reasons. The cement manufacturer is invited to use them in order to improve his cement. These materials are being sold under dozens of trade names.

Nine years ago the author presented a paper[2] before this Society dealing with this subject. Since 1920 new types of admixtures have been placed on the market and other changes in concrete materials and methods seem to make a new study of this subject desirable.

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[1] Director of Research, International Cement Corp., New York City.

[2] "Effect of Hydrated Lime and Other Powdered Admixtures in Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. XX, Part II, p. 149 (1920).

The two series of concrete tests described in this paper were carried out during the winter and spring of 1929 in the Research Laboratory of the International Cement Corporation, New York City. The present interest in early strength of concrete caused us to include tests at 1, 2 and 3 days. The unusual fineness of grinding which characterizes the high-early-strength cements was the reason for carrying out most of the tests on a commercial portland cement which was ground to a fineness of 98 per cent through the No. 200 sieve.

This investigation has to do principally with studies of effect of admixtures on workability and strength of concrete. The 7 commercial admixtures tested were selected as representatives of the usual types. Seven non-commercial materials were included in order to extend the scope of the tests. The concrete tested was of the type generally used in road construction.

We have under way a comprehensive study of both inert and chemically active admixtures with particular reference to their influence on volume change in cement mixtures. This study will be reported upon later.

OUTLINE OF TESTS

Series 18 was of a preliminary nature and somewhat restricted in scope.

Ten 6 by 12-in. cylinders were made from each batch of machine-mixed concrete, which consisted of 100 lb. of sand and 170 lb. of gravel. The quantities of cement, water and admixture are given in Table I. In the *basic mix* for comparison with admixtures we used 50 lb. of cement. This was about 1:1.6:3.0 in terms of cement, sand and gravel; water-ratio 0.73.

Flow and slump tests were made on each batch. We endeavored to control the flow to 180 by adjusting the quantity of mixing water (except in reference Nos. 5 and 18). Two cylinders were tested at 1, 2 and 3 days, one each at 7 and 28 days and 3 months. In general, only one batch was mixed of each kind; however, for reference Nos. 14 to 18, two batches were mixed.

The following admixtures were used with two types of portland cement:

(A)	Hydrated lime	10 per cent
(C)	Colloidal clay (treated)	2, 5, 10 "
(D)	Colloidal clay	5 "
(E)	Colloidal clay	5 "
(G)	Tripoli	10 "
(H)	Volcanic ash	10 "

The admixture replaced an equal weight of cement.

Several batches of concrete were mixed without admixtures with the following purpose in view.

1. To determine the effect on concrete strength of small additions of cement.

2. To determine the effect on strength and workability from using the same quantities of cement and water as in the case of 5 per cent of admixture C, but omitting the admixture (reference Nos. 5 and 18).

3. To determine the effect on strength and workability from adding cement to the same cost as 5 per cent of admixture C (reference Nos. 2 to 4 and 15 to 17).

Series 18A was more comprehensive and consisted of two major divisions.

1. Tests of concrete without admixture, but using five consistencies each of the following mixes:

1:2.6:4.6
1:2.0:3.5
1:1.7:3.0
1:1.6:2.8
1:1.5:2.6
1:1.3:2.3

and one consistency of the following mixes:

1:3.5:6.2
1:1.2:2.0

In the first group above, the quantity of mixing water was varied in such a manner as to secure flows ranging from about 135 to 230 (slump ½ to 9 in.).

2. Tests of 1:1.7:3.0 concrete (1:4) in which cement was replaced by 3 to 5 percentages of the following admixtures:

(A) Hydrated lime
(B) Limestone
(C) Colloidal clay (treated)
(D) Colloidal clay
(E) Colloidal clay
(F) Moler clay (calcined)
(H) Volcanic ash
(J) Slag
(K) Precipitator dust
(L) Artificial admixture
(M) Artificial admixture
(N) Artificial admixture
(O) Diatomaceous earth

The tests under the first group above gave definite information for these combinations of cement and aggregates on the following topics:

1. Effect of changes in cement content on strength and workability of concrete.
2. Effect of changes in quantity of mixing water on strength and workability of concrete.
3. Quantity of water required to produce a given flow when cement content was increased (see discussion of "water factor").

MATERIALS AND CONCRETE

Both series of concrete tests were made from the same materials and as nearly as practicable under the same conditions. Before beginning these tests the men employed on this work had mixed about 500 similar batches; 266 batches were mixed in this investigation.

Cement.—Two cements were used:

1. Standard portland cement, a mixture of equal parts of 4 brands of commercial cements manufactured in the vicinity of Chicago.
2. High-early-strength portland cement, ground to 98 per cent through the No. 200 sieve.

The cements conformed to all the requirements of the A.S.T.M. Specifications. Briquet tests are not reported, since it is believed that the concrete tests will more satisfactorily indicate the properties of the cements. Fineness and normal consistency are given in Table IV.

Aggregates.—The aggregates consisted of sand and gravel from Long Island, graded to about 1¼ in. They were used in a room-dry condition.

Mixing Water.—The mixing water was from New York City supply. Care was used to have the water at 70° F.

Mixing.—The concrete was mixed in a 3½-cu.-ft. tilting mixer. All materials were proportioned by weight.

Workability.—The workability of the concrete was measured by the flow and slump tests. A 30-in. flow table was used, fifteen ½-in. drops. The flow is expressed as a percentage of the original cone diameter of 10 in. Slump tests were made in accordance with the Society's Tentative Method of Test for Consistency of Portland-Cement Concrete (D 138 - 26 T).[1]

Molding Cylinders.—The entire batch was used in molding 6 by 12-in. cylinders—10 from a batch in Series 18; 14 in Series 18A.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part I, p. 874 (1926); also 1928 Book of A.S.T.M. Tentative Standards, p. 518.

Steel molds were used. Each mold stood on a planed cast-iron plate. A similar plate was used to form the top cap.

Curing Concrete.—The cylinders remained in the molds for 18 to 22 hours. They were then placed in the moist room where they were cured at 70° F. until tested.

Concrete Tests.—The cylinders were tested in a damp condition in a 300,000-lb. screw-power machine.

Sequence of Tests.—In Series 18A one batch of each kind was tested before making the duplicate batches. After going entirely through the program twice the triplicate batches of each kind were mixed.

Admixtures.—Fourteen admixtures were used. The 7 commercial materials were secured from the manufacturers or dealers. The trade names are not reported, but it is believed that the designations will indicate the types of materials used. The colloidal clays were of the Bentonite type.

The fineness, unit weight, normal consistency (as determined by the Vicat needle) of the admixtures are given in Table IV.

In determining fineness, it was necessary to wash some of the materials through the sieve.

Test Data

The data of concrete tests are given in Tables I, II and III.

The effect of the admixtures on strength is summarized in Table IV. Figures 1 to 17 show the principal properties of the concrete and the effect of admixtures on strength and workability.

Discussion of Concrete Tests

Inter-Relation of Quantities of Cement and Water in Batch and Resulting Water-Ratio, Flow and Strength.—Before discussing the effect of powdered admixtures, it is necessary to have a clear conception of the properties of similar concrete without admixtures. In making concrete we start with the three constituent materials and end with certain desired properties which may be characterized as strength (or other measures of the ultimate quality), workability and yield.

If we represent mixing water by W, cement by C, aggregate by A, strength by S, yield by Y and flow by F, we may show these relations by the following scheme:

$$\begin{array}{ccc} \begin{array}{c}\text{BEGIN WITH}\\ \text{CONSTITUENT}\\ \text{MATERIALS}\end{array} & & \begin{array}{c}\text{END WITH}\\ \text{DESIRED}\\ \text{PROPERTIES}\end{array} \\ \left.\begin{array}{r} W \\ C \\ A \end{array}\right\} & = & \left\{\begin{array}{l} S \text{ (or } W/C\text{)} \\ Y \text{ (or Cost)} \\ F \end{array}\right. \end{array}$$

For the purpose of this discussion the flow will be considered a satisfactory measure of workability and strength as a measure of quality.

The present discussion will be confined to the *quantities* of constituent materials and disregard the matter of quality.

Many investigators have studied the influence of the various factors on strength, so that the principles governing the inter-relation of strength, quantities of cement, water, size and grading of aggregates, etc., are fairly well understood. The water-cement ratio law[1] has been so thoroughly established that we may consider the water-cement ratio as a measure of concrete quality as reflected by strength, permeability, resistance to destructive agencies, etc.

Workability is an essential property of concrete and is just as important as strength or other desired properties. Whether we realize it or not we pay for workability in the same coin as for strength and durability. If we buy more workability than we need, we are either wasting money or subtracting so much from concrete quality. Workability and strength may be considered as complementary properties of concrete.

While the laws of concrete strength and yield are understood, only fragmentary data are available on the influence of changes in quantities of constituent materials on workability.

In this investigation special care was taken to secure data on the inter-relation of quantities of cement and water on workability as measured by the flow test. Since the aggregates were the same in all tests we can give data for only this combination of materials. The inter-relation of cement, water, water-cement ratio, flow and 1-day strength are shown in Fig. 9. It is believed that this is the first attempt to bring out these relations in a single diagram. Many of the fundamental relations which are pointed out in the following discussion may be traced in this figure. The diagram shows a number of minor inconsistencies, since the tests were not sufficient to fix all points definitely, yet the general trends of the values are unmistakable.

Water-Cement Ratio - Strength Relation.—The fundamental relation between quantity of mixing water (expressed in terms of volume of cement) and the resulting concrete strength is brought out by both series of tests, see Figs. 1 and 4. The high-early-strength portland cement gave a family of water-cement ratio curves of the same type as the standard cement; the tests at 1, 2, 3 and 7 days gave curves of the same form as at 28 days. The mathematical expression for the water-cement ratio law and the constants for this cement are given in the notes under Fig. 4.

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[1] Originally published in author's "Design of Concrete Mixtures," *Bulletin 1*, Structural Materials Research Laboratory, Lewis Institute, Chicago (1918). See notes following Fig. 4.

The influence of water-cement ratio on concrete strength at ages from 1 to 28 days is shown in another way in Fig. 5, which was plotted directly from the curves in Fig. 4.

Flow Test as a Measure of Workability.—A wide range of flows was covered by the concrete without admixtures. In the admixture tests a flow of 180 was aimed at.

The consistent relation found between flow and water-cement ratio and flow and strength between limits 135 to 230 show that for the materials used in these tests the flow table offers a satisfactory measure of workability. It is notable that the flows were more consistent near the outer limits than elsewhere.

For these materials there was a nearly constant relation between flow and slump for values between flow 170 to 230 and slump 2 to 9 in., as shown in Fig. 17. Each 1-in. change in slump corresponds to a change of 8 in flow table units. The relation between slump and strength, at ages 1, 3 and 28 days is shown in Fig. 16.

Relation Between Flow and Strength of Concrete.—It was stated above that workability and strength of concrete are complementary properties; both are essential and whatever is added to one (with fixed proportions of cement and aggregates) is removed from the other. These factors deserve a more complete consideration than is permissible in this paper. Figure 11 shows the relation between strength and flow for 1-day concrete tests of high-early-strength cement. These curves and also those for other ages show essentially a linear relation between flow and strength for a uniform quantity of cement; the strength falls off rapidly with increased flow. The loss in compressive strength for an increase of 10 in flow (using 68-lb. batch high-early-strength cement and a flow of 180 as a basis for comparison) was as follows:

AGE AT TEST, DAYS	LOSS IN STRENGTH LB. PER SQ. IN.	LOSS IN STRENGTH PER CENT
1	170	8
2	190	6
3	220	6
7	240	5
28	270	5

For leaner mixtures the percentage loss in strength was greater than those shown; the losses were about the same for richer mixtures.

The relation between flow and strength for 1-day tests is shown also in Fig. 9.

Effect on Concrete Strength of Small Changes in Cement Content.—Figures 2 and 6 show how the strength of concrete is influenced by changes in the cement content. Separate curves are given for each age. The figures in parentheses on the tangents show the increase in concrete strength for each 1 per cent increase in cement, using 50 lb. of cement in Fig. 2 and 68 lb. in Fig. 6 as a basis for comparison. The values are tabulated below:

	PERCENTAGE INCREASE IN STRENGTH DUE TO ADDING 1 PER CENT OF CEMENT					
	1 DAY	2 DAYS	3 DAYS	7 DAYS	28 DAYS	3 MONTHS
Standard Portland Cement...	1.0	1.0	0.9	0.9	0.8	0.7
High-Early-Strength Portland Cement..................	1.4	1.2	1.1	1.0	0.9	...

The above values refer to the *basic mixes*, similar to those with admixtures. The addition of cement has a greater effect on the early strength than at 28 days or 3 months.

If we plot the quantity of cement to a logarithmic scale the curves in Fig. 6 become essentially straight lines; see Fig. 7. In other words, concrete strength for a fixed proportion of sand and gravel and a uniform flow is *proportional to the logarithm of the quantity of cement.* This is only another way of saying that the quantity of water required to give a fixed flow is directly proportional to the quantity of cement (as is shown in Fig. 9). The strength is directly proportional to the logarithm of the quantity of water required to produce a given flow with increasing quantities of cement. It should be noted that these statements do not conflict with the water-cement ratio - strength law.

A given increase in cement has a much greater effect in increasing strength of lean mixtures than the richer ones. The well-known law of diminishing return seems to apply to this case. This fact sets a distinct limitation on the economy of obtaining high strengths by means of rich mixtures.

Effect on Strength of Small Changes in Mixing Water.—Two distinct cases must be considered:

1. Additions of water to a fixed quantity of cement with increasing flow and increasing water-cement ratio.

2. Additions of water on account of increasing cement content, with uniform flow and decreasing water-cement ratio.

The inter-relations of these two cases are clearly shown in Fig. 9. Case 1 is the most common and the one of most interest in the present discussion. The following values calculated from Figs. 1 and 4 illustrate the effect on strength of small changes in mixing water when the quantity of cement is constant:

		LOSS IN CONCRETE STRENGTH FOR EACH 1 PER CENT INCREASE IN MIXING WATER				
		1 DAY	2 DAYS	3 DAYS	7 DAYS	28 DAYS
Standard Portland Cement	lb. per sq. in..	18	25	27	35	46
	per cent......	1.9	1.4	1.2	1.2	1.1
High-Early-Strength Portland Cement...........	lb. per sq. in..	42	50	52	57	63
	per cent......	1.9	1.5	1.4	1.3	1.2

The foregoing values were derived from concretes of the "basic mix" in each case or about 1:1.7:3.0, water-cement ratio 0.80.

The loss in strength in pounds per square inch is higher at later ages than at early periods and is greater for the high-early-strength cement than for standard; however, in terms of the percentage change in strength, the values are essentially the same for both cements, ranging from about 2 per cent at 1 day to 1.1 per cent at 28 days. It appears that the percentages will decrease gradually at later ages, although direct data are not now available.

The loss in strength due to 1 per cent increase in mixing water is about 40 to 50 per cent greater than the gain in strength from adding 1 per cent of cement.

Little consideration need be given to case 2, except to point out that the addition of water due to increasing cement, results in a rapid decrease in the water-cement ratio for a given flow and consequently greatly increased strength. All of these cases are only different phases of the water-cement ratio law.

Effect on Flow of Small Changes in Cement Content.—Figure 9 shows how the flow is affected by changes in cement content, based on tests of high-early-strength cement; we did not secure data which would permit such a study of standard portland cement.

For a given water-cement ratio, flowability is increased about 2.0 flow units for each 1 per cent added cement. For a given water content, flowability is decreased about 1.0 flow units for each 1 per cent added cement.

Effect on Flow of Small Changes in Mixing Water.—Figure 9 shows that for a given cement content, the flow increases with the water-cement ratio. Each 1 per cent increase in mixing water corresponds to an increase in flowability of 3.5 flow units, using 68 lb. cement, water 34 to 40 lb. as a basis for computation. See also Fig. 8.

DISCUSSION OF TESTS OF POWDERED ADMIXTURES

The effect of powdered admixtures in concrete will be discussed principally from the following standpoints:

1. Effect of admixture on strength,
2. Effect of admixture on workability,

3. Endeavor to reconcile the observed effects in terms of known properties of the concrete and admixtures.

The first feature to strike the attention in testing concrete with admixtures is that in general more mixing water is required to produce concrete of a given flow. This leads to a closer inquiry as to the reason for this additional water and to a comparison of the effects of similar water additions if admixtures were not present.

The quantity of mixing water required for concrete seems to be dictated by the following considerations:

1. Forming a plastic mass of the dry materials,
2. Hydration of cement,
3. Absorption of aggregates and cement.

The exact quantity of water required for each of these purposes is not well known but depends on many factors such as relative quantities of cement and aggregates, size and grading of aggregates, workability desired, etc.

In the actual making of concrete the quantity of water required for hydration is only incidental and is generally small as compared with the quantity necessary for workability. The water necessary for workability and absorption is also available for hydration (provided it is not allowed to escape prematurely by improper curing).

In view of the importance of quantity of water a special inquiry was made of the water requirements of the admixtures.

With the same materials and with a narrow range of mixtures, how much water must be added to a concrete mix in order to maintain the same flow when either the cement is increased or powdered admixtures are used? The answer to this question will furnish the key to the effect of admixtures on flow and strength. This leads to the introduction of a new term—the "water factor" of concrete materials.

The admixtures used *replaced an equal weight* of cement; consequently it was necessary to secure accurate data on the effect of small changes in cement content on the properties of concrete without admixtures.

It should be emphasized that the conclusions from these tests would have been exactly the same if the admixtures had been *added* to a constant quantity of cement.

Water Factor.—The water factor of concrete materials may be defined as the quantity of mixing water required by the addition of a unit weight of any constituent in order to maintain a given workability. This term may be applied to any concrete material, hence it seems best to express the water in terms of percentage by weight of the material under consideration.

Tests were made on concrete with a wide range of cement quantities using the same weights of sand and gravel, hence the water factor for this cement may be calculated; this can best be done by reference to the flow lines in Fig. 9. The values ranged from 11 per cent for a flow of 130 to 22 per cent for a flow of 230. For a flow 180 the water factor was 15 per cent; in other words, 0.15 lb. of water must be added for each 1 lb. of cement.

Tests not included in this report gave water factors of 11 per cent for the sand and 4 per cent for the gravel for a flow of 180.

It is interesting to note that the water factor for cement in the lower range of plastic mixtures is only one-half to two-thirds the normal consistency.

The water factor for the admixtures is given in Table IV. These values were computed directly from data in Tables I and III, and take into account the fact that the admixture replaced an equal weight of cement. The water factor ranged from 0 for slag and limestone to 195 and 238 per cent for the colloidal clays. The intimate relation between the water factors of the admixtures and their normal consistencies will be pointed out below.

If we use 5 per cent of an admixture with a water factor of 200 per cent, we must add 9.4 lb. of water for each sack of cement; this is as much water as would be required by 86 lb. of sand or 235 lb. of gravel. In other words, by omitting the admixture and using the same water-cement ratio we could increase the yield of concrete per sack of cement by 20 to 25 per cent without any sacrifice of strength or workability. This gives another measure of the high price we must pay for the use of certain admixtures.

Effect of Admixture on Strength.—The foregoing discussion has indicated the intimate relation between the quantity of mixing water and the strength and flow of concrete. These relations are of major importance when discussing the effect of powdered admixtures.

The method of computing the effect of admixtures on strength was as follows:

The strength ratios in Tables I and III were subtracted from 100 per cent giving the total change in concrete strength for a given percentage of admixture and age. This quantity was divided by the percentage of admixture in terms of cement.

In case different percentages of admixture were used, the changes, expressed as a percentage of the concrete strength without admixture were averaged. Since 1 per cent of cement was replaced by each 1 per cent of admixture, it was necessary to deduct from the percentage change in strength due to admixture the percentages for 1 per cent of cement which are given at the top of Table IV. This gives the *net change* in strength (in per cent) due to each 1 per cent of admixture.

The effect was approximately the same, regardless of the percentage of admixture used, however, the higher percentages in Table III were not considered in computing the net change in Table IV.

Where parallel tests were made, the effect of admixtures was essentially the same for both types of cement.

The effect of admixtures can be computed graphically as shown in Fig. 12. The solid curve shows the 1-day strength of plain concrete and may be considered as an enlargement of a part of the curve in Fig. 6. The heavy dotted curves give the corresponding strengths for diatomaceous earth and for colloidal clay (treated). The difference in the lengths of the vertical ordinates represents the loss in strength for a given percentage of admixture. The average value computed in this way for colloidal clay is 5.8 per cent as compared with 5.7 per cent given in Table IV where the effect was calculated directly from the test data.

Figure 13 gives strength-age curves for the admixtures using the values from Tables I, II and III. The top curve for each group represents cement only, and shows directly the effect of admixtures, and enables us to compare the strengths of the two cements. Only typical curves have been drawn in the upper group, but the strengths for each admixture may be identified by the characteristic points which are marked in Fig. 14.

Figure 14 shows the net change in concrete strength for each 1 per cent of the admixture used, also for the two cements.

The following admixtures have slight hydraulic value, in other words, tend to set and harden in the presence of cement and water:

(J) Slag,
(K) Precipitator dust,
(N) Artificial admixture.

These materials showed a small increase in concrete strength, especially at early ages. It will be noted that the water factors of these materials are low.

All the other admixtures were non-hydraulic and without exception showed reduction in strength of concrete. The reduction was more pronounced at early ages. There is an intimate relation between the reduction in concrete strength and their normal consistencies and the water factors as shown in Fig. 15.

An example of the serious reduction in concrete strength by two of the admixtures is shown in Fig. 11. The principal items involved are given also in the following table. The flow was approximately 180 in all cases:

	Quantities in Batch			Amount of Cement which Gave the Same 1-day Strength without Admixture, lb.
	Admixture, lb.	Cement, lb.	Water, lb.	
Admixtures				
Colloidal Clay (treated)...	2	66	41.7	58
	5	63	47.0	46
	10	58	56.0	39
Diatomaceous Earth......	1	67	38.7	66
	2	66	40.2	62
	5	63	44.5	53

These values illustrate the disastrous effect of such admixtures on concrete strength. The colloidal clay does not show much improvement with age; the reduction in strength from diatomaceous earth is confined largely to early periods.

In most of the tests the admixtures replaced an equal weight of cement. It will be interesting to determine directly the effect of adding cement to the same cost as the admixture. Such a study was made with colloidal clay (treated). The cost of replacing 1 lb. cement with colloidal clay would pay for about 3½ lb. of cement. The details of quantities used are given in Table I. The concrete strengths for standard cement are plotted in Fig. 3. The following typical values were taken from the curves:

	Concrete Strength, lb. per sq. in.					
	1 Day	2 Days	3 Days	7 Days	28 Days	3 Months
95 per cent of cement and 5 per cent of admixture.....	500	1100	1500	2100	3300	3800
Cement same cost as above..	1200	2200	2600	2700	5000	5900

The concrete with 5 per cent colloidal clay had at 1 day less than half the strength of similar concrete when the same sum was spent on cement only; at 3 months the strength was 64 per cent of that for cement only. A similar study may be made for the high-early-strength cement.

Effect of Admixtures on Workability.—None of the admixtures had any beneficial effect on the workability of concrete as measured by the flow test.

The influence of flow and water-cement ratio are shown in Fig. 8. This diagram has been referred to above, but we wish now to call attention to the points for various percentages of the admixtures. A material *increase in mixing water must be made in order to maintain the original flow.* If the flow were improved, the admixtures points would have been above and to the left of the 68-lb. cement line, Both series of tests and both cements show the same relation in this respect.

In the preceding discussion it was shown that most of the admixtures required an excess of water to maintain the flow which was obtained without the admixture. In other words, if the water had not been increased the flow would have been *reduced by the admixture.* Since the strength and other properties of concrete are materially benefited by reducing the flow by simply reducing the mixing water, there would seem to be no place for admixtures for this purpose.

The exact influence of 4 typical admixtures on flow and strength are shown in Fig. 9.

Replacing cement by an equal weight of admixtures C, O or A required much additional water to maintain a flow of 180, consequently their curves cut the curves for higher water-cement ratios (and lower strengths). Admixture B required only a slight increase in water, consequently the strength was approximately that of the cement. Five per cent of admixture C required 47 lb. of mixing water (water-cement ratio 1.12) in order to produce a flow of 180; if this water had been used with the cement only (63 lb.) we would have secured a flow of about 240 and approximately the same concrete strength (see values for these quantities in Figs. 10 and 4).

It is a strange commentary on the unsatisfactory state of the present knowledge of this subject, that one of the producers of admixtures emphasizes as a special merit, the high water factor of his material—the very property which makes this admixture fatal to both workability and strength of concrete.

By using concrete tests in order to study the effect of a non-hydraulic powdered admixture, we seem to be adopting a very indirect and laborious method of determining its normal consistency.

All of the commercial admixtures included in these tests must be characterized as reducing both the flow and strength of concrete.

CONCLUSIONS

The following conclusions may be stated with reference to powdered admixtures in concrete:

The workability of the concrete as measured by the flow test was not improved by any of the 14 powdered admixtures tested.

The water factor required to produce the same workability as similar concrete without admixture was approximately proportional to the normal consistency of the admixture as measured by the Vicat needle.

All of the commercial admixtures tested must be classed as injurious adulterants, due to their high water factors. The reduction in concrete strength was approximately proportional to their water factor.

The colloidal clays of the Bentonite type were most injurious on account of their high water factor as reflected also by high normal consistencies. Loss in strength ranged from about 1.5 per cent to 5 per cent for each 1 per cent of admixture in terms of weight of cement.

After the colloidal clays, diatomaceous earth was the most injurious to strength and workability, although the serious loss in strength was confined to early ages.

The slightly hydraulic admixtures, powdered slag (high calcium granulated), precipitator dust, and one of the artificial admixtures showed slight increase in strength at early ages, but this benefit was doubtful after ages of 7 days.

In general, loss in strength due to admixtures was greater at ages of 1 to 3 days than at ages of 7 days to 3 months.

The effect of admixtures was independent of the quantity of admixtures used, although quantities higher than 15 per cent were not considered.

Reference may be made to the text for a number of important conclusions drawn from a study of strength and workability of plain concrete as influenced by changes in quantities of cement and mixing water.

TABLE I.—EFFECT OF POWDERED ADMIXTURES ON CONCRETE STRENGTH—SERIES 18.

Compression tests of 6 by 12-in. concrete cylinders.
1:4 Concrete; for 50 lb. of cement per batch.
Sand and gravel from Long Island; graded to 1¼ in. 100 lb. sand and 170 lb. gravel in each batch.
Admixture mixed with dry cement by thorough rolling on canvas.
Machine-mixed concrete; 10 cylinders from each batch; 2 tested at 1, 2, 3 days; 1 at 7 days, 28 days and 3 months.
Water-cement ratios computed for cement only.
Values in this table based on one batch of each kind.

Reference Number	Quantities of Materials						Flow Value	Slump, in.	Compressive Strength, lb. per sq. in.						Strength Ratio[a]					
	Cement, lb.	Admixture			Water															
		Kind	Per Cent	Pounds	Pounds	Water-Cement Ratio			1 Day	2 Days	3 Days	7 Days	28 Days	3 Months	1 Day	2 Days	3 Days	7 Days	28 Days	3 Months
STANDARD PORTLAND CEMENT																				
1	50	None	..	...	24.7	0.74	180	2	950	1820	2210	2910	4260	5530	100	100	100	100	100	100
2[b]	53½	None	..	...	26.0	0.73	185	2¾	1120	2040	2450	3510	4570	5370	118	112	111	121	107	97
3[b]	58½	None	..	...	26.5	0.68	190	3¼	1180	2120	2540	3360	4800	6010	124	116	115	116	113	109
4[b]	68½	None	..	...	28.0	0.62	182	3	1360	2530	2950	4250	5830	6510	143	139	134	147	137	108
5[c]	47½	None	..	...	32.0	1.01	245	7½	470	980	1300	2020	3100	3910	50	54	59	70	73	71
6	45	Hydrated Lime.....(A)	10	5	26.0	0.87	180	3	720	1050	1980	2050	3890	3940	76	58	89	71	91	71
7	49	Colloidal Clay (treated)..... (C)	2	1	29.5	0.90	180	1¾	740	1350	1800	2530	3880	4500	78	74	81	87	91	81
8	47½	(C)	5	2½	32.0	1.01	180	2¼	530	1120	1520	2120	3230	3740	56	62	69	73	76	67
9	45	(C)	10	5	34.0	1.13	180	2¼	360	770	1010	1440	2380	3070	38	43	45	50	56	55
10	47½	Colloidal Clay......(D)	5	2½	27.5	0.87	180	2¼	800	1450	1980	2830	4180	4880	84	80	89	97	98	88
11	47½	Colloidal Clay......(E)	5	2½	30.5	0.96	180	2¼	580	1300	1530	2250	3430	4250	61	71	69	72	80	77
12	45	Tripoli.............(G)	10	5	28.0	0.93	180	1¾	670	1430	1790	2580	3930	5010	70	79	81	87	92	91
13	45	Volcanic Ash.......(H)	10	5	27.0	0.90	185	3½	710	1530	1750	2510	3890	4810	75	84	79	86	91	87
HIGH-EARLY-STRENGTH PORTLAND CEMENT																				
14[d]	50	None	..	...	24.7	0.74	182	2¾	2270	3660	4210	5210	6120	5890	100	100	100	100	100	100
15	53½	None	..	...	26.0	0.73	180	3½	2460	3820	4260	5450	6150	6770	109	104	101	105	101	115
16	58½	None	..	...	26.0	0.66	180	2¾	2940	4140	4870	6380	....	7260	130	113	116	123	...	123
17	68½	None	..	...	27.5	0.60	180	5¼	3110	4210	4770	....	6750	....	137	115	113	...	112	...
18[d]	47½	None	..	...	30.3	0.96	222	8	1250	2410	2660	3470	3780	4420	55	66	63	66	62	75
19	45	Hydrated Lime.....(A)	10	5	26.0	0.87	182	4	2090	3430	3720	4670	5310	5290	92	92	88	90	87	90
20[d]	49	Colloidal Clay (treated)..... (C)	2	1	27.0	0.83	181	2½	2070	3390	3680	4740	5270	5650	91	92	87	91	86	96
21[d]	47½	(C)	5	2½	30.2	0.95	180	2½	1550	2560	2780	3450	4200	4280	68	70	66	66	69	73
22[d]	45	(C)	10	5	36.0	1.20	180	2¾	880	1850	1960	2470	2690	2860	39	50	47	47	44	49
23	47½	Colloidal Clay......(D)	5	2½	26.8	0.85	180	3¾	2000	3040	3360	3670	5250	4420	88	83	80	70	86	75
24	47½	Colloidal Clay......(E)	5	2½	29.3	0.93	180	2¼	1600	2600	2850	3610	4050	4480	71	71	68	69	66	76
25	45	Tripoli.............(G)	10	5	26.0	0.87	185	4	1730	2830	3390	4460	5080	5380	76	77	80	86	83	92
26	45	Volcanic Ash.......(H)	10	5	27.0	0.90	180	5	1540	2720	3770	3890	4890	5020	68	74	90	75	80	85

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[a] Based on concrete strength of same cement without admixture, using 50 lb. cement per batch.

[b] No admixture, but added cement equal to cost of 2, 5 or 10 per cent of colloidal clay (treated) as in Nos. 7 to 9 and 20 to 22.

[c] No admixture, but same quantities of cement and water as No. 8 and 18, with 5 per cent of colloidal clay (treated).

[d] Average of 2 batches.

TABLE II.—TESTS OF CONCRETE WITHOUT ADMIXTURES—SERIES 18A.

Compression tests of 6 by 12-in. concrete cylinders.
High-early-strength portland cement.
Sand and gravel from Long Island.
Each batch consisted of 140 lb. sand and 240 lb. gravel and the quantities of cement and water in the table.
Machine-mixed concrete; 14 cylinders from each batch; 2 tested at each age.
Strength values are the average of 6 cylinder tests—2 from each of 3 batches mixed on different days.

Reference Number	Cement, lb.	Mix	Water		Flow Value	Slump, in.	Compressive Strength, lb. per sq. in.					Strength Ratios				
			Pounds	Water-Cement Ratio			1 Day	2 Days	3 Days	7 Days	28 Days	1 Day	2 Days	3 Days	7 Days	28 Days
27	33	1:3.5:6.2	34.0	1.55	175	1½	480	800	1060	1640	1900	22	24	27	37	36
28	44	1:2.6:4.6	30.0	1.03	140	½	1280	2340	2750	3290	3860	59	70	72	74	73
29			33.7	1.14	168	2	1010	1900	2230	2760	3190	46	57	59	62	62
30			35.9	1.22	190	4	850	1650	2040	2570	2990	39	50	54	58	57
31			38.5	1.31	207	6	730	1560	1870	2470	2900	33	47	49	56	55
32			41.0	1.40	232	9	600	1250	1610	2140	2640	27	38	42	48	50
33	58	1:2:3.5	32.8	0.85	135	½	1980	3110	3700	4460	4980	90	94	97	101	94
34			36.5	0.95	173	2	1610	2640	2980	3620	4130	74	80	79	81	88
35			38.5	1.00	190	4	1360	2310	2690	3310	3950	62	70	71	75	75
36			40.5	1.05	203	6	1210	2070	2440	2920	3540	55	62	64	66	67
37			44.0	1.14	227	9	960	1710	2120	2710	3330	44	51	56	61	63
38	68	1:1.7:3.0	33.7	0.75	137	½	2800	3730	4350	5180	5970	128	112	115	117	113
39[a]			36.2[a]	0.80[a]	177[a]	2¼[a]	2190[a]	3320[a]	3800[a]	4440[a]	5280[a]	100	100	100	100	100
40			38.5	0.85	188	4	1960	3210	3400	3990	4880	90	97	90	90	92
41			42.0	0.93	207	6	1640	2590	3040	3640	4210	75	78	80	82	80
42			45.7	1.01	222	9	1250	2110	2510	3210	3930	57	63	66	72	74
43	73	1:1.6:2.8	34.3	0.71	135	½	2830	3900	4530	5460	6190	129	118	119	121	117
44			37.5	0.77	175	2	2410	3560	3930	4710	5600	110	107	103	106	106
45			40.2	0.83	194	4¼	2030	3260	3560	4560	4950	93	98	94	103	94
46			42.7	0.88	207	6	1780	2590	3260	3990	4670	81	78	85	90	88
47			46.7	0.96	230	9	1380	2360	2770	3390	3780	63	71	73	76	72
48	78	1:1.5:2.6	35.0	0.68	137	½	2810	4270	4770	5800	6500	128	128	126	131	123
49			37.8	0.73	168	2	2460	3940	4310	5510	6030	113	119	113	124	114
50			41.5	0.80	188	4	1930	3290	3970	4780	5240	88	99	105	108	99
51			44.8	0.86	207	6¼	1490	2780	3250	4120	4720	68	84	86	93	89
52			48.2	0.93	230	9	1210	2330	2870	3720	4100	55	70	75	84	77
53	88	1:1.3:2.3	37.2	0.63	143	¾	3140	4190	4640	5760	6690	143	126	122	130	126
54			39.5	0.67	167	2	2990	4050	4600	5710	6170	137	122	121	129	117
55			42.7	0.73	187	4	2450	3620	4210	5130	5850	112	109	111	116	111
56			45.5	0.78	208	6¼	2130	3230	3770	4660	5480	98	98	99	105	104
57			48.8	0.83	227	9	1650	2680	3060	3910	4600	75	84	81	88	87
58	100	1:1.2:2.0	39.0	0.59	180	2¼	3560	4430	5140	5800	6650	163	133	135	131	125

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[a] Average of 10 batches (20 cylinders). These values were used as 100 per cent in computing strength ratios in this table and as a basis of comparison in studying the effect of admixtures in Table III.

TABLE III.—TESTS OF CONCRETE WITH POWDERED ADMIXTURES—(SERIES 18A).

Concrete mixed during same period and from same materials as that in Table II.
Compression tests of 6 by 12-in. concrete cylinders.
Mix 1:1.7:3.0.
High-early-strength portland cement.
Sand and gravel from Long Island, graded to $1\frac{1}{4}$ in.
Powdered admixtures replaced equivalent weight of cement.
Admixture mixed with dry cement by thorough rolling on canvas.
Machine-mixed concrete; 14 cylinders from each batch; 2 tested at each age.
Each batch consisted of 140 lb. of sand and 240 lb. of gravel and the quantities of cement and water given in the table.
Strength values are the average of 6 cylinder tests—2 from each of 3 batches mixed on different days.
Water-cement ratios were computed on basis of cement only, that is, the admixtures were not considered.

Reference Number	Cement, lb.	Admixture		Water		Flow Value	Slump, in.	Compressive Strength, lb. per sq. in.					Strength Ratios[a]				
		Pounds	Per Cent	Pounds	Water-Cement Ratio			1 Day	2 Days	3 Days	7 Days	28 Days	1 Day	2 Days	3 Days	7 Days	28 Days
HYDRATED LIME (A)																	
59	66	2	2.9	37.5	0.85	180	$2\frac{1}{4}$	2040	3460	3980	5010	5460	93	102	105	113	103
60	63	5	7.4	39.5	0.94	180	$2\frac{1}{4}$	1800	3080	3530	4320	4810	82	91	93	97	91
61	58	10	14.7	41.5	1.08	180	3	1450	2600	2900	3700	4330	66	77	78	83	82
62	48	20	29.4[b]	45.0	1.41	185	4	1000[b]	1900[b]	2130[b]	2870[b]	3250[b]	46[b]	56[b]	56[b]	65[b]	61[b]
LIMESTONE (B)																	
63	66	2	2.9	36.7	0.83	180	$2\frac{1}{4}$	1850	3030	3610	4320	5080	84	90	95	97	96
64	63	5	7.4	37.2	0.89	180	$2\frac{1}{2}$	1710	2920	3350	4290	5150	78	86	88	96	97
65	58	10	14.7	36.2	0.97	180	3	1400	2520	3130	3810	4890	64	75	82	86	92
66	48	20	29.4[b]	37.2	1.17	180	$2\frac{1}{4}$	1170[b]	2250[b]	2800[b]	3250[b]	3920[b]	53[b]	66[b]	74[b]	73[b]	74[b]
COLLOIDAL CLAY (TREATED) (C)																	
67	66	2	2.9	41.7	0.95	182	$2\frac{1}{4}$	1570	2720	3110	4000	4600	72	80	82	90	87
68	63	5	7.4	47.0	1.12	180	$2\frac{1}{4}$	1050	2110	2500	3100	3500	48	62	66	70	66
69	58	10	14.7	56.0	1.45	180	$2\frac{3}{4}$	645	1270	1640	2110	2400	29	37	43	47	45
COLLOIDAL CLAY (D)																	
70	66	2	2.9	38.0	0.87	180	$2\frac{3}{4}$	1740	3210	3740	4520	5100	79	95	98	102	97
71	63	5	7.4	40.5	0.96	180	$2\frac{1}{2}$	1570	2870	3390	4040	4550	72	85	89	91	86
72	58	10	14.7	43.0	1.11	180	$2\frac{1}{2}$	1160	2160	2550	3250	3870	53	64	67	73	73
MOLER CLAY (CALCINED) (F)																	
73	66	2	2.9	37.8	0.86	180	$2\frac{1}{4}$	1930	3220	3600	4710	5170	88	95	95	106	97
74	63	5	7.4	39.8	0.95	180	$2\frac{1}{4}$	1700	2960	3450	4180	4920	78	87	91	94	93
75	58	10	14.7	43.7	1.13	180	$2\frac{1}{2}$	1220	2280	2860	3550	4600	56	70	75	80	87

[a] Based on 1:1.7:3 mix, water-cement ratio 0.80, flow 177, see No. 39, Table II.

[b] These values for the higher percentages of admixtures were not used in computing the net change in concrete strength in Table IV.

TABLE III.—TESTS OF CONCRETE WITH POWDERED ADMIXTURES—(SERIES 18A) (*Continued*).

Reference Number	Cement, lb.	Admixture		Water		Flow Value	Slump, in.	Compressive Strength, lb. per sq. in.					Strength Ratios[a]				
		Pounds	Per Cent	Pounds	Water-Cement Ratio			1 Day	2 Days	3 Days	7 Days	28 Days	1 Day	2 Days	3 Days	7 Days	28 Days
VOLCANIC ASH (H)																	
76	66	2	2.9	37.6	0.86	180	2¾	2020	3350	3610	4390	5270	92	99	95	99	100
77	63	5	7.4	38.0	0.91	180	3¼	1840	3200	3490	4110	5030	84	95	92	92	95
78	58	10	14.7	39.0	1.01	180	2¾	1510	2710	3110	3750	4150	69	80	82	85	88
SLAG (J)																	
79	66	2	2.9	36.5	0.83	182	3¼	2040	3390	3800	4260	5450	93	100	100	96	103
80	63	5	7.4	37.7	0.90	180	2¾	2110	3190	3370	4120	4860	96	94	88	93	92
81	58	10	14.7	37.2	0.97	180	2½	1620	2860	3280	3880	5020	74	85	86	88	95
82	48	20	29.4[b]	36.5	1.14	180	2½	1230[b]	2230[b]	2490[b]	3440[b]	4430[b]	56[b]	66[b]	65[b]	77[b]	84[b]
83	34	34	50.0	36.5	1.62	180	2¼	500[b]	1200[b]	1630[b]	2280[b]	4020[b]	23[b]	35[b]	43[b]	51[b]	76[b]
PRECIPITATOR DUST (K)																	
84	66	2	2.9	36.7	0.83	180	2¾	1940	3350	3920	4660	5590	89	99	103	105	105
85	63	5	7.4	37.2	0.89	180	2¾	2120	3320	3490	4690	5190	97	98	92	106	98
86	58	10	14.7	37.2	0.97	183	3	2100	3390	3700	4260	4990	96	100	97	96	95
87	48	20	29.4[b]	40.0	1.25	181	2¾	1890[b]	2910[b]	3350[b]	3700[b]	4560[b]	86[b]	86[b]	88[b]	83[b]	86[b]
88	34	34	50.0[b]	41.5	1.84	182	3	725[b]	1840[b]	2240[b]	3000[b]	3970[b]	33[b]	54[b]	59[b]	68[b]	75[b]
ARTIFICIAL ADMIXTURE (L)																	
89	66	2	2.9	37.7	0.86	180	2¼	1850	2690	3910	4560	5310	85	80	103	104	99
90	63	5	7.4	39.9	0.95	182	2¾	1750	3050	3400	4450	4910	80	90	89	100	93
91	58	10	14.7	42.1	1.09	180	3	1510	2620	3260	3850	4840	69	77	86	87	91
ARTIFICIAL ADMIXTURE (M)																	
92	66	2	2.9	38.4	0.87	181	2¾	1830	3310	3680	4470	5180	83	98	97	101	98
93	63	5	7.4	40.5	0.97	180	2¾	1580	3180	3510	4300	4810	72	94	92	97	91
94	58	10	14.7	43.1	1.12	180	2¼	1450	2840	3370	3960	4760	66	84	89	89	90
ARTIFICIAL ADMIXTURE (N)																	
95	66	2	2.9	37.8	0.86	180	2½	2180	3440	3880	4740	5210	100	102	102	107	99
96	63	5	7.4	39.5	0.94	180	2½	2060	3190	3570	4610	5180	94	94	94	104	98
97	58	10	14.7	41.5	1.08	180	2½	1990	3110	3630	4350	5050	91	92	96	96	95
98	48	20	29.4[b]	45.5	1.42	180	2¼	1090[b]	2070[b]	2480[b]	3380[b]	4200[b]	50[b]	61[b]	65[b]	76[b]	80[b]
99	34	34	50.0	51.0	2.26	180	2½	640[b]	1090[b]	1290[b]	2500[b]	3360[b]	29[b]	32[b]	34[b]	56[b]	64[b]
DIATOMACEOUS EARTH (O)																	
100	67	1	1.5	38.7	0.87	180	2½	1980	3210	3520	4350	5300	91	95	92	98	100
101	66	2	2.9	40.2	0.92	180	2¼	1790	2960	3190	4240	5180	82	88	84	95	98
102	63	5	7.4	44.5	1.06	180	2½	1390	2420	3040	3590	4730	64	71	80	81	90

[a] Based on 1:1.7:3 mix, water-cement ratio 0.80, flow 177, see No. 39, Table II.

[b] These values for the higher percentages of admixtures were not used in computing the net change in concrete strength in Table IV.

TABLE IV.—SUMMARY OF EFFECT OF POWDERED ADMIXTURES IN CONCRETE.

Net change in strength computed from strength-ratios in Tables I and III. Percentages of admixtures above 15 per cent were not used (see note, Table II). The fact that the admixture replaced an equal weight of cement was taken into account in computing net change; see text.

Corresponding values for cements are given for comparison.

"Water Factor" is quantity of water which must be added to batch in order to maintain a flow of 180, expressed as a percentage of weight of added cement or admixture.

Admixture	Fineness, through No. 200 Sieve	Unit Weight, lb. per cu. ft.	Normal Consistency, per cent	Data from Table	Cement	Water Factor, per cent	Net Change in Concrete Strength for Each 1 per cent Admixture by Weight					
							1 Day	2 Days	3 Days	7 Days	28 Days	3 Months
Standard Portland Cement	84	94	23	I	Standard	15[a]	+1.0	+1.0	+0.9	+0.9	+0.8	+0.7
High-Early-Strength Portland Cement	98	94	25	II	High-Early-Strength	15	+1.4	+1.2	+1.1	+1.0	+0.9	.....
Hydrated Lime (A)	96	34	62	I	Standard	10	−1.4	−3.2	−0.2	−2.0	−0.1	−2.2
				I	High-Early-Strength	10	+0.9	+0.4	−0.7	−0.9	−0.6	−0.3
				III	High-Early-Strength	45	−0.9	+0.5	+0.9	0	+0.4	.....
					Average	22	−0.5	−0.7	0	−1.0	−0.1	−1.3
Limestone (B)	78	89	20	III	High-Early-Strength	0	−2.5	−1.1	−0.4	+0.2	+0.1	.....
Colloidal Clay (treated) (C)	90	54	350	I	Standard	300	−7.8	−7.8	−6.2	−4.8	−3.8	−5.3
				I	High-Early-Strength	225	−4.3	−3.8	−5.1	−4.5	−5.4	−3.3
				III	High-Early-Strength	190	−5.7	−4.2	−3.8	−2.7	−3.4	.....
					Average	238	−5.9	−5.3	−5.0	−4.0	−4.2	−4.3
Colloidal Clay (D)	75	58	120	I	Standard	94	−2.1	−3.0	−1.3	+0.2	+0.8	−1.7
				I	High-Early-Strength	68	−1.0	−1.4	−2.9	−5.0	−1.8	−4.1
				III	High-Early-Strength	59	−3.4	−0.9	−0.4	+0.2	−0.7	.....
					Average	73	−2.2	−1.8	−1.5	−1.5	−0.6	−2.9
Colloidal Clay (E)	73	86	220	I	Standard	210	−6.8	−4.8	−5.3	−4.7	−3.2	−3.9
				I	High-Early-Strength	180	−4.4	−4.6	−5.3	−5.2	−5.9	−3.9
					Average	195	−5.6	−4.7	−5.3	−5.0	−4.6	−3.9
Moler Clay (Calcined) (F)	90	32	81	III	High-Early-Strength	56	−2.0	−0.6	−0.4	+1.0	0	.....
Tripoli (G)	90	56	54	I	Standard	50	−2.0	−1.1	−1.0	−0.5	0	−0.3
				I	High-Early-Strength	10	−1.0	−1.1	−0.9	−0.4	−0.8	−0
					Average	30	−1.5	−1.1	−0.9	−0.4	−0.4	−0.2
Volcanic Ash (H)	80	64	58	I	Standard	76	−1.5	−0.6	−1.2	−0.5	−0.1	−0.7
				I	High-Early-Strength	76	−2.0	−1.4	+0.1	−1.5	−1.1	−0.6
				III	High-Early-Strengt	30	−0.9	+0.4	−0.2	+0.2	+0.4	.....
					Average	61	−1.5	−0.5	−0.4	−0.6	−0.3	−0.7
Slag (J)	92	85	20	III	High-Early-Strength	0	+0.1	+0.6	+0.2	+0.1	−0.8	.....
Precipitator Dust (K)	94	54	48	III	High-Early-Strength	2	+0.1	+1.0	+1.0	−0.3	−0.5	.....
Artificial Admixture (L)	90	37	76	III	High-Early-Strength	53	−2.0	−2.0	+0.6	−0.8	+0.3	.....
Artificial Admixture (M)	83	30	104	III	High-Early-Strength	72	−2.5	+0.3	+0.2	+0.7	0	.....
Artificial Admixture (N)	85	37	65	III	High-Early-Strength	50	+0.9	+0.9	+0.9	−0.2	+0.6	.....
Diatomaceous Earth (O)	95	13	183	III	High-Early-Strength	180	−4.4	−2.6	−3.4	−0.9	+0.2	.....

[a] Value was not determined for standard cement, used same as for high-early-strength cement.

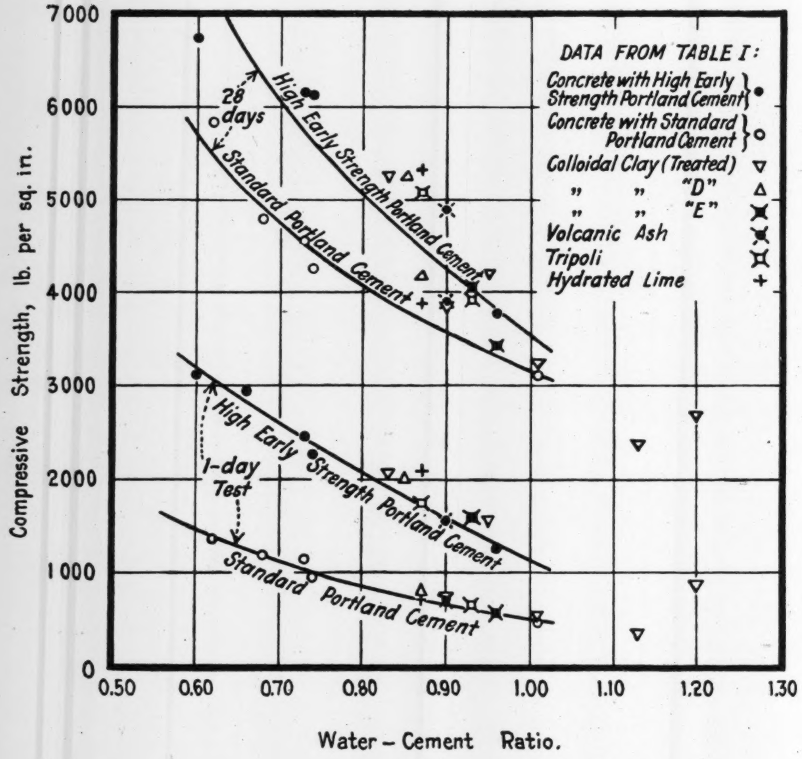

FIG. 1.—Strength - Water-Cement Ratio Curves for Concrete Tests in Table I.

Compression tests of 6 by 12-in. cylinders.
Standard and high-early-strength portland cements.
Water-cement ratio computed for cement only.
Flow about 180 (slump 2 to 3 in.).
Curves plotted for 1 and 28 days to points for concrete without admixtures; similar curves may be drawn for other ages.
Compare Fig. 4.

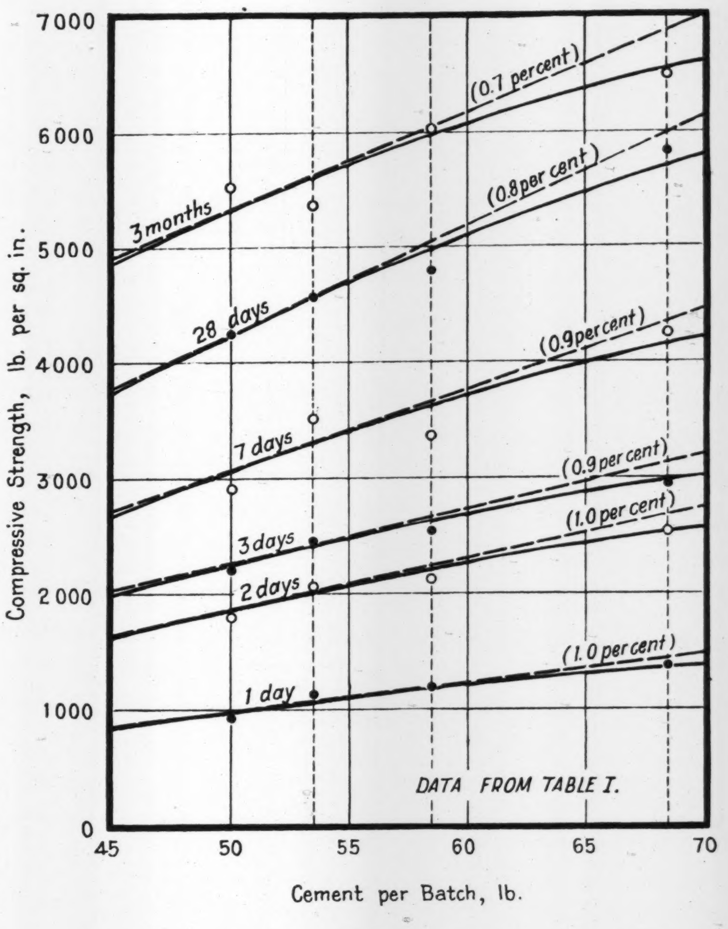

FIG. 2.—Strength - Cement Curves for Standard Portland Cement.

Compression tests of 6 by 12-in. cylinders.

Flow about 180 (slump 2 to 3 in.).

The figures on the tangents are the increase in concrete strength for each 1 per cent added cement, based on the 50-lb. mix.

Compare Fig. 6.

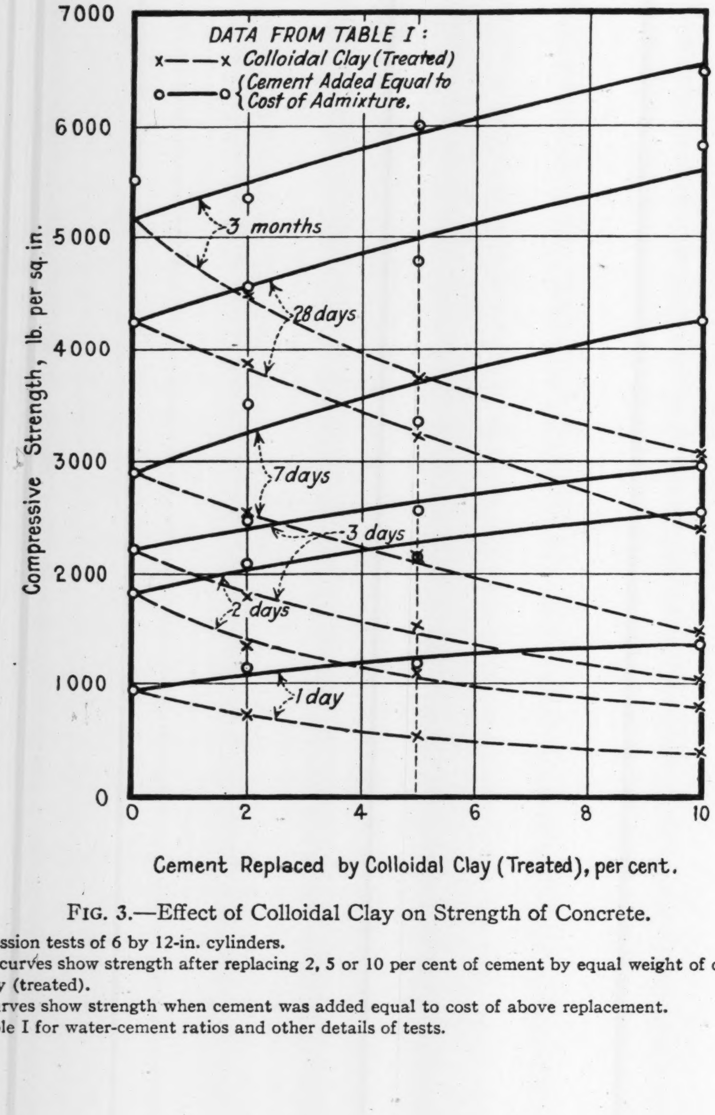

FIG. 3.—Effect of Colloidal Clay on Strength of Concrete.

Compression tests of 6 by 12-in. cylinders.

Dotted curves show strength after replacing 2, 5 or 10 per cent of cement by equal weight of colloidal clay (treated).

Solid curves show strength when cement was added equal to cost of above replacement.

See Table I for water-cement ratios and other details of tests.

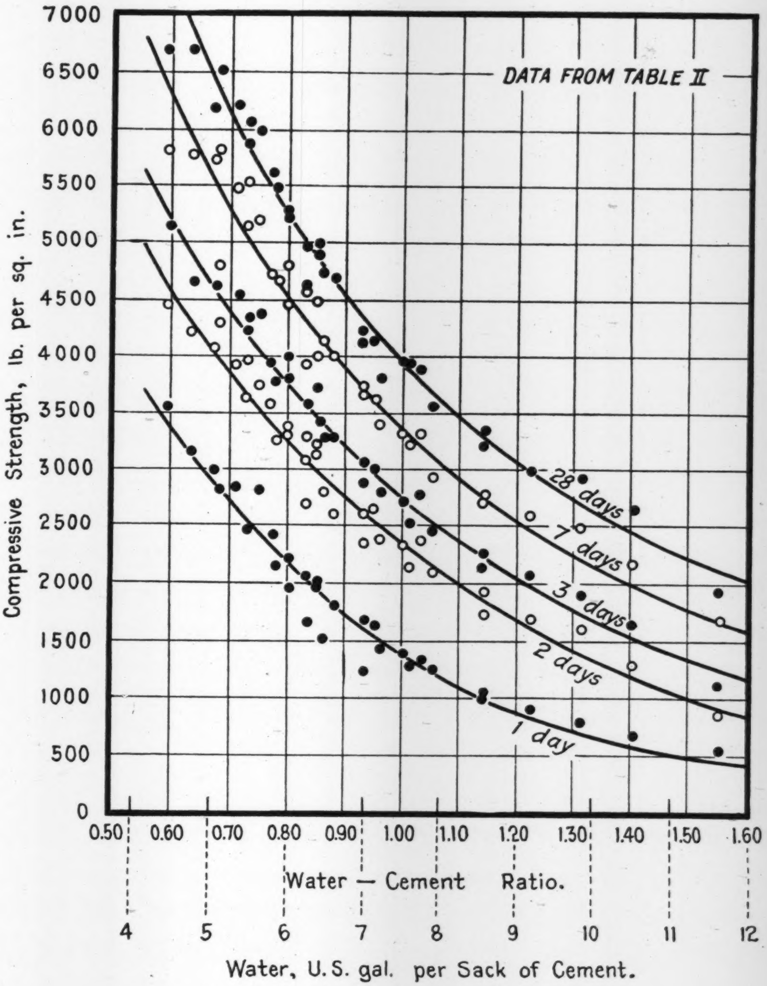

FIG. 4.—Strength - Water-Cement Ratio Curves for High-Early-Strength Portland Cement.

Compression tests of 6 by 12-in. cylinders.

Based on tests of 8 mixtures, 6 of which occur in 5 different degrees of workability (flow varied from 135 to 230; slump $\frac{1}{2}$ to 9 in.).

These curves furnish the basis for the water-cement ratio law which may be expressed as

$$S = \frac{A}{B^x}$$

where S = compressive strength of concrete

A and B are constants whose values depend on type of cement and test conditions

x = (an exponent) = Water-cement ratio (ratio of volume of water to volume of cement in the batch, exclusive of that absorbed by aggregates).

For these tests A = 15,000 lb. per sq. in. and appears to be independent of the age of the concrete.

B = 11 at 1 day; 6.6 at 2 days; 5.6 at 3 days; 4.4 at 7 days and 3.7 at 28 days.

Compare Fig. 1.

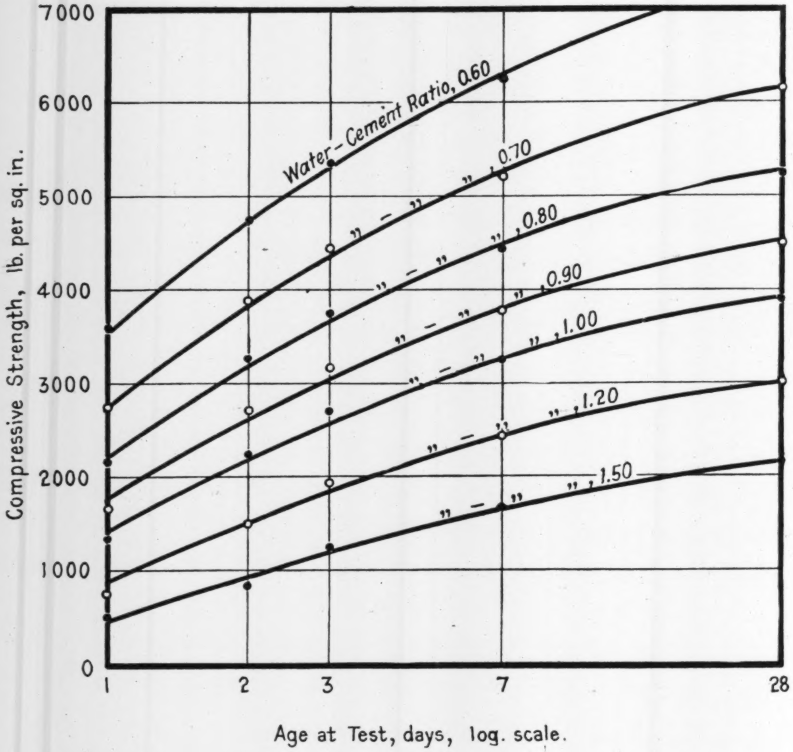

FIG. 5.—Strength - Age Curves for High-Early-Strength Portland Cement.

Plotted from strength curves in Fig. 4.

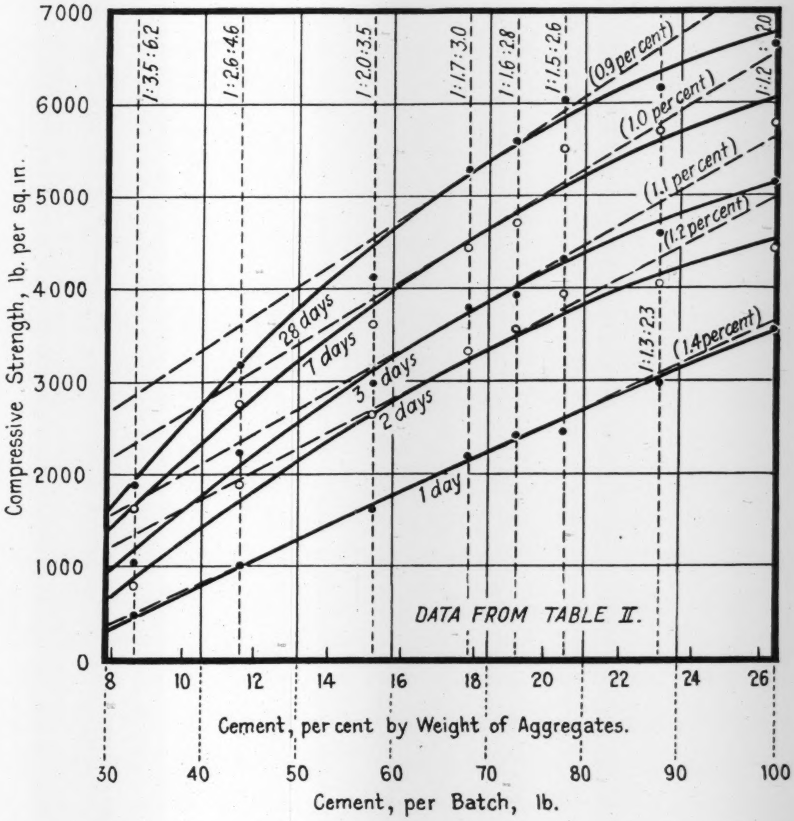

FIG. 6.—Strength - Cement Curves for High-Early-Strength Portland Cement.

Compression tests of 6 by 12-in. cylinders.

Flow about 180.

The figures on the tangents are the increase in concrete strength for each 1 per cent added cement, based on the 1:1.7:3.0 mix (68 lb. cement).

Compare Figs. 2 and 7.

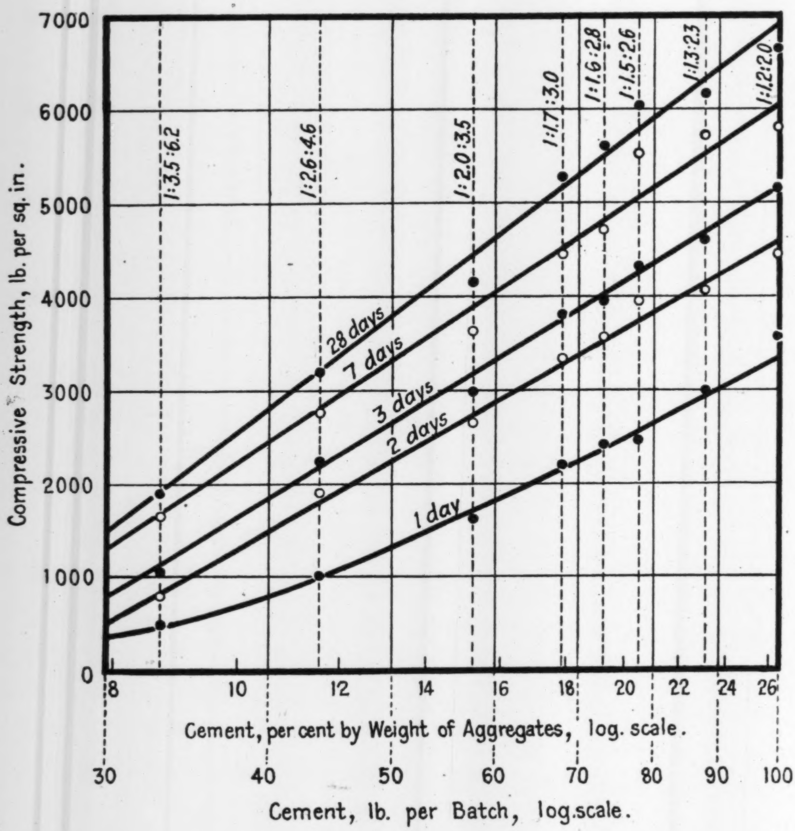

FIG. 7.—Strength - Cement Curves for High-Early-Strength Portland Cement.
Same curves as Fig. 6, except that cement is plotted to logarithmic scale.

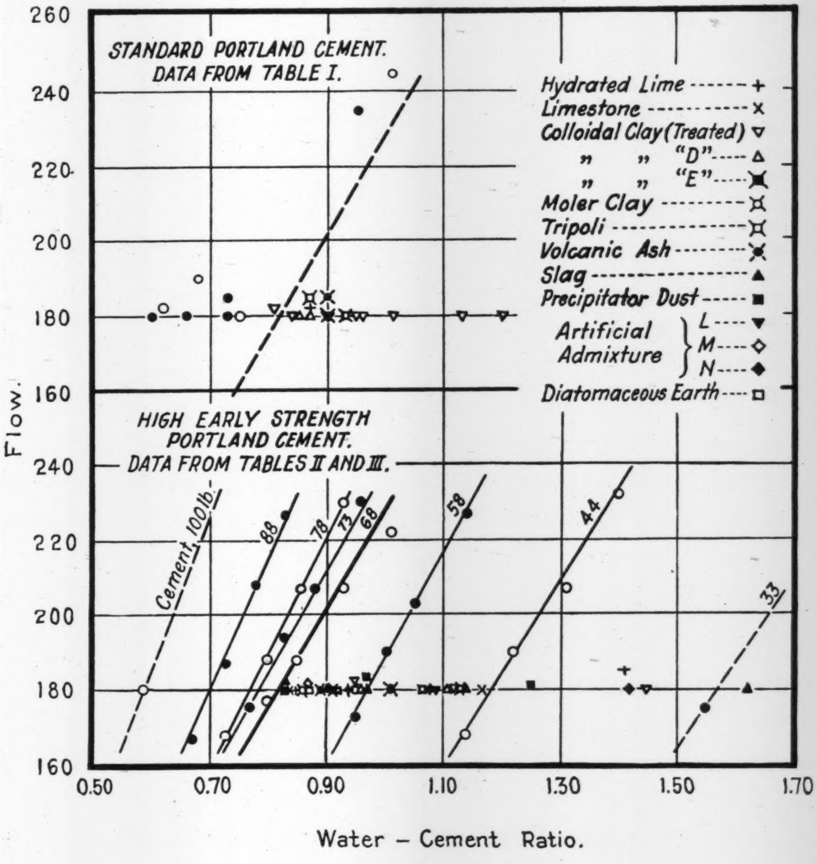

FIG. 8.—Flow - Water-Cement Ratio Curves.

Show that admixtures require increased water-cement ratio in order to maintain flow about 180. Water-cement ratio for admixtures should be compared with line for 68-lb. cement in lower figure. Dotted curve in upper figure drawn parallel to 68-lb. curve in lower figure.

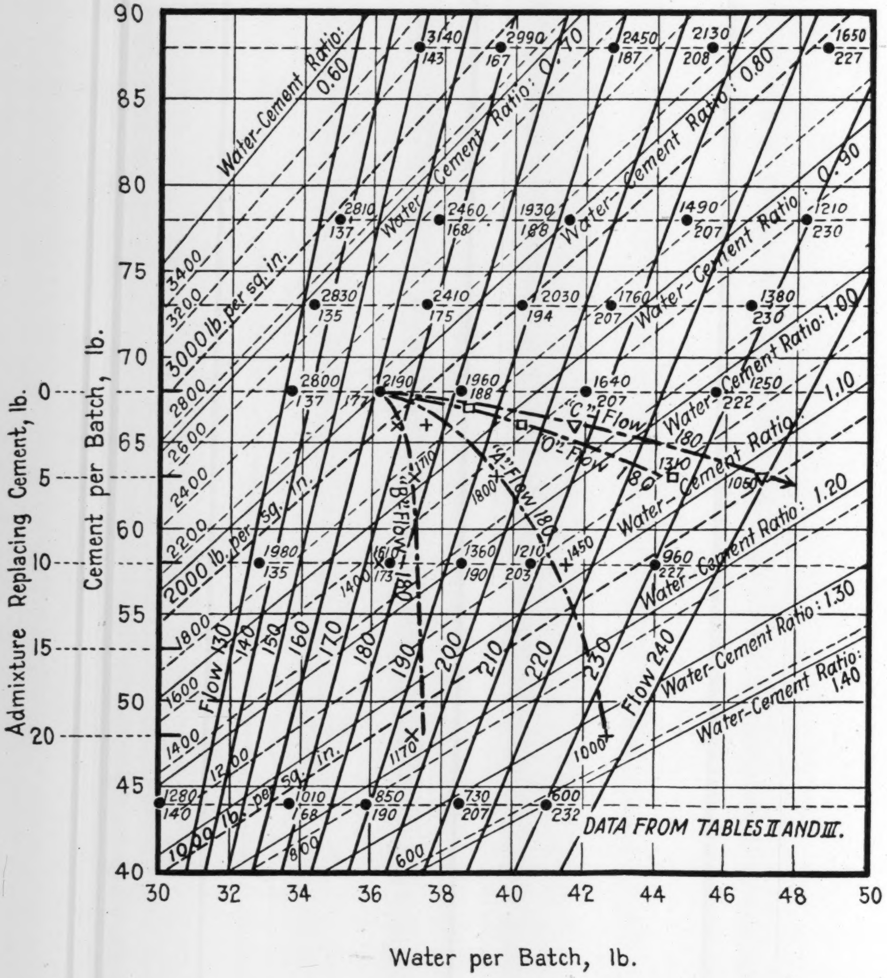

FIG. 9.—Relation Between Cement, Water, Flow, Water-Cement Ratio and Strength of High-Early-Strength Portland Cement Concrete.

This figure gives many of the fundamental relations for concrete without admixtures.

The 1-day compressive strengths (of high-early-strength cement) and flow are given for each point; similar curves may be plotted for other ages.

The following admixtures are included (flow 180) to illustrate their effect on quantity of mixing water and strength:

A, Hydrated lime,
B, Powdered limestone,
C, Colloidal clay (treated),
O, Diatomaceous earth.

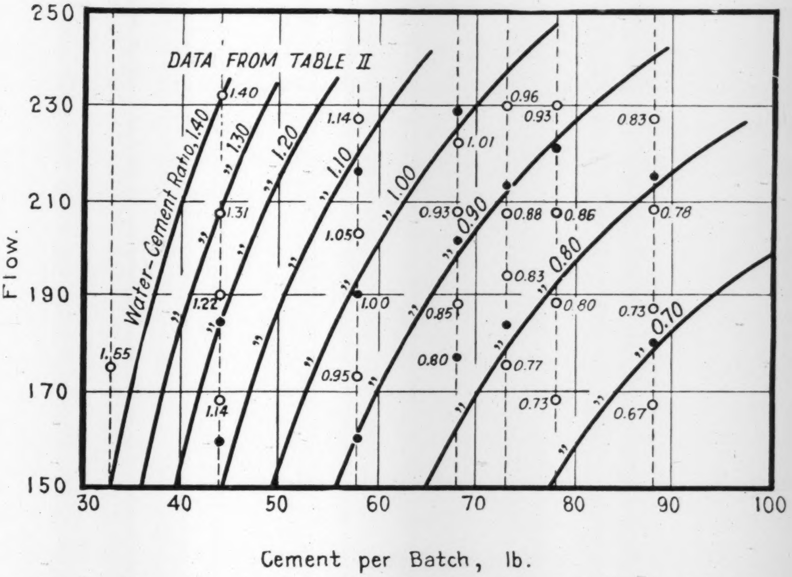

FIG. 10.—Flow - Cement Curves for High-Early-Strength Portland Cement.

The values opposite the open circles are the flows obtained in the tests; the solid circles were transferred from the curves in Fig. 8. Same data shown in Fig. 9.

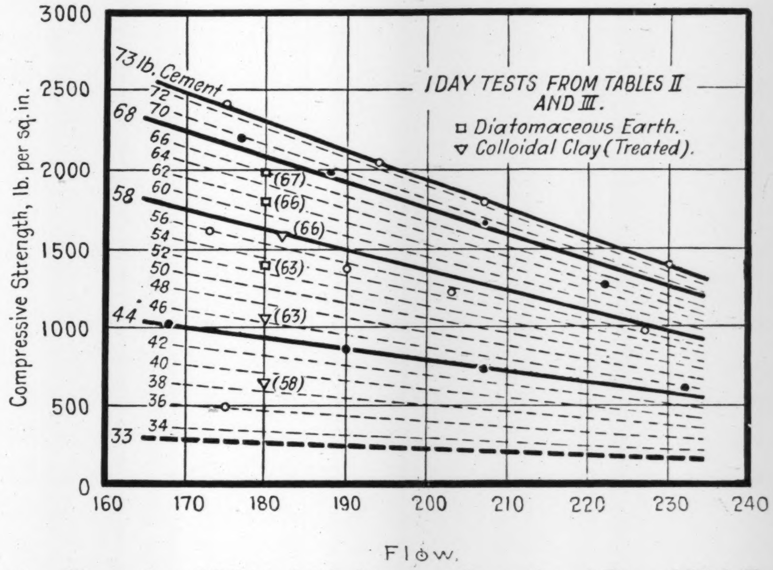

FIG. 11.—Illustration of Effect of Admixtures on Strength and Flow of High-Early-Strength Portland Cement Concrete.

The values in parentheses show actual pounds of cement used with the admixture; these may be compared with the pounds of cement which when used alone give the same flow and strength.

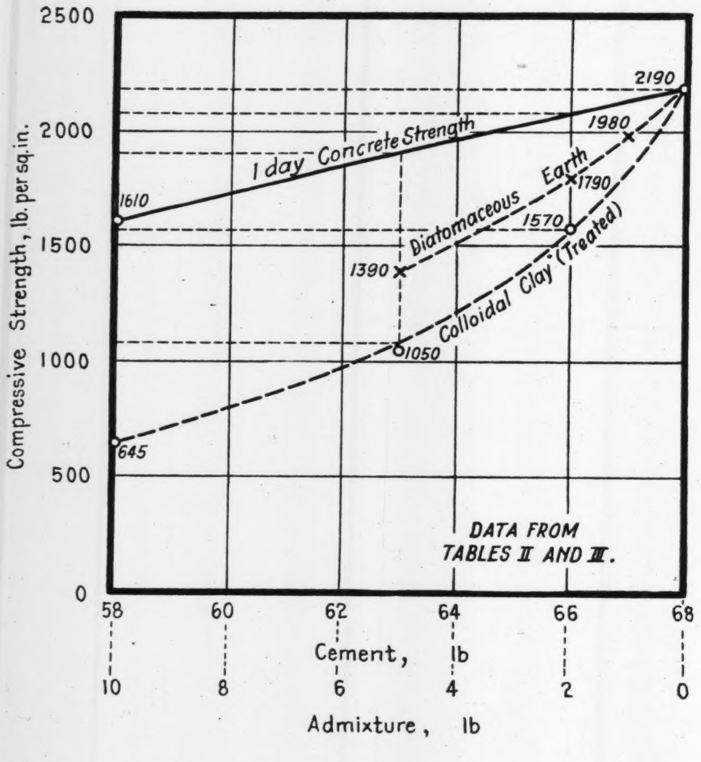

FIG. 12.—Graphical Method of Computing Effect of Admixtures on Concrete Strength.

1-day compressive strengths of high-early-strength cement for the quantities of cement and admixture shown.

The difference in the lengths of the vertical ordinates represents the loss in strength due to the admixtures.

If admixture had been added to a fixed quantity of cement (68 lb.) the curve for plain concrete would have been brought to a horizontal position, and the admixture curves would have been raised proportionally; the net effect would have been the same.

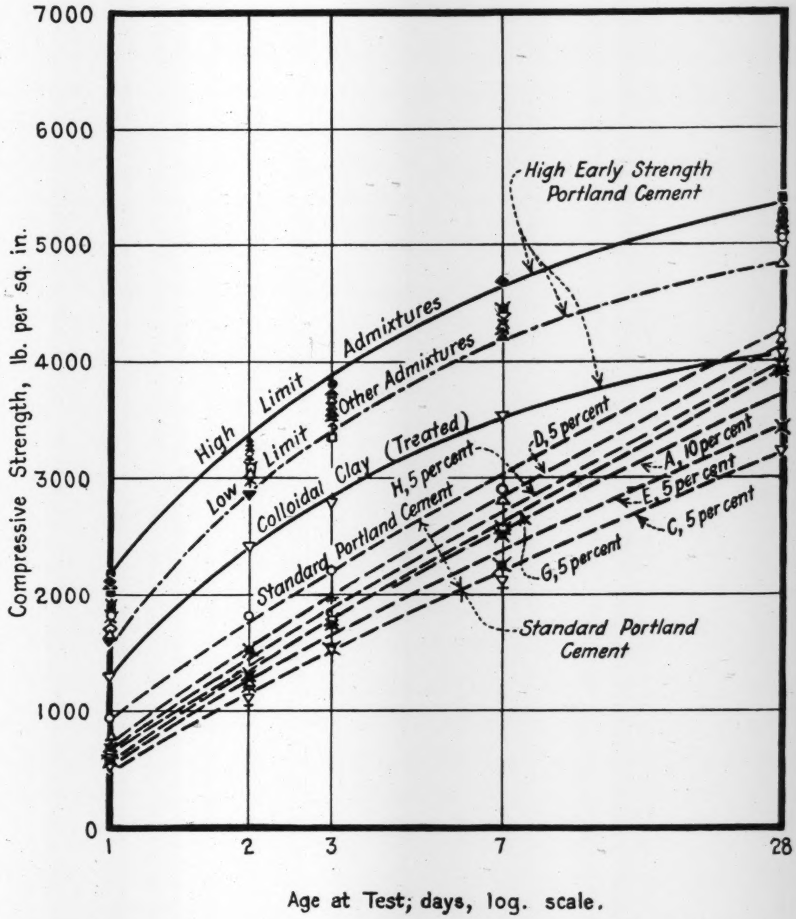

FIG. 13.—Strength - Age Curves for Concrete With and Without Admixture.

Compression tests of 6 by 12-in. concrete cylinders.

Mix about 1:1.7:3.0, flow about 180.

Lower group of curves based on tests with standard portland cement from Table I; upper group based on high-early-strength portland cement from Tables II and III.

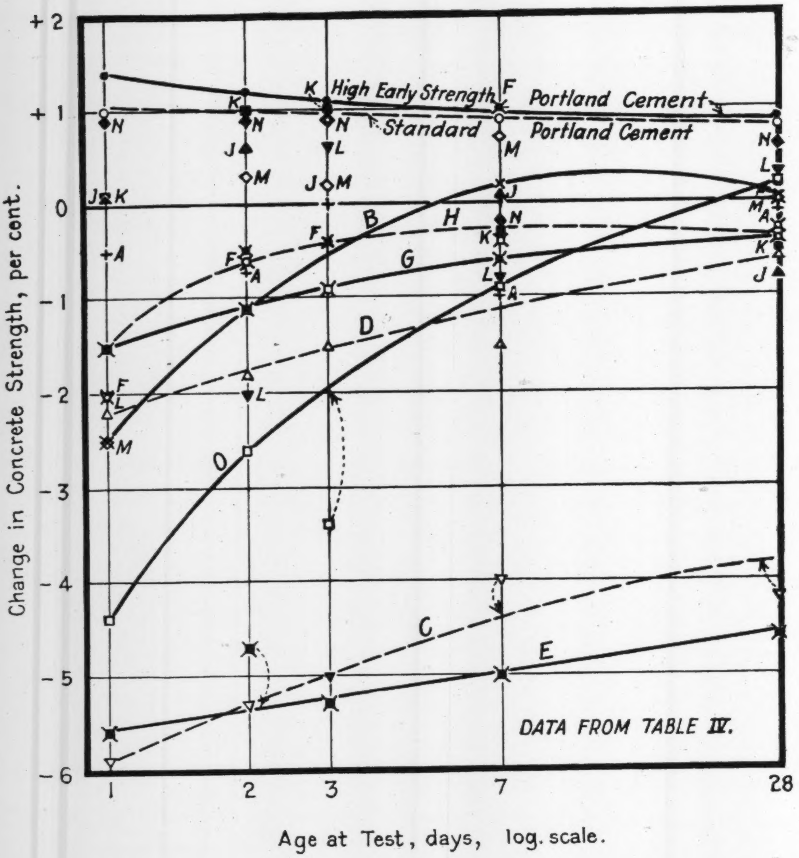

FIG. 14.—Change in Concrete Strength by 1 per cent of Cement or Admixture.

Net changes as explained in text.

A,	Hydrated lime	H,	Volcanic ash
B,	Limestone	J,	Slag
C, D, E,	Colloidal clay	K,	Precipitator dust
F,	Moler clay (calcined)	L, M, N,	Artificial Admixtures
G,	Tripoli	O,	Diatomaceous earth

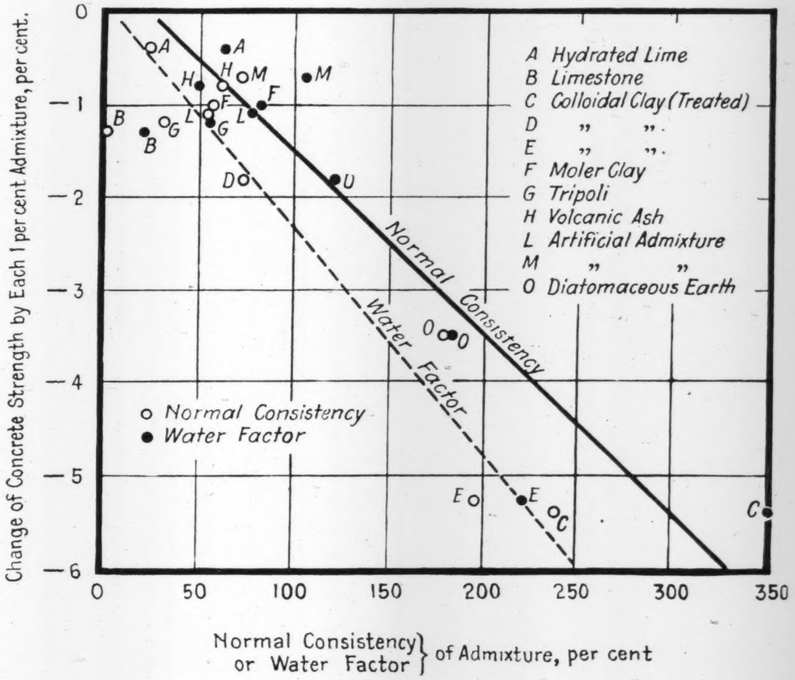

FIG. 15.—Relation of Normal Consistency and Water Factor of Admixtures to Loss in Concrete Strengths.

Based on average of 1, 2 and 3-day tests of non-hydraulic admixtures only.
Normal consistency determined by Vicat needle as in standard cement tests.
Water factor is quantity of mixing water, expressed as a percentage, which must be added for each unit of weight of admixture in order to bring flow up to original value.

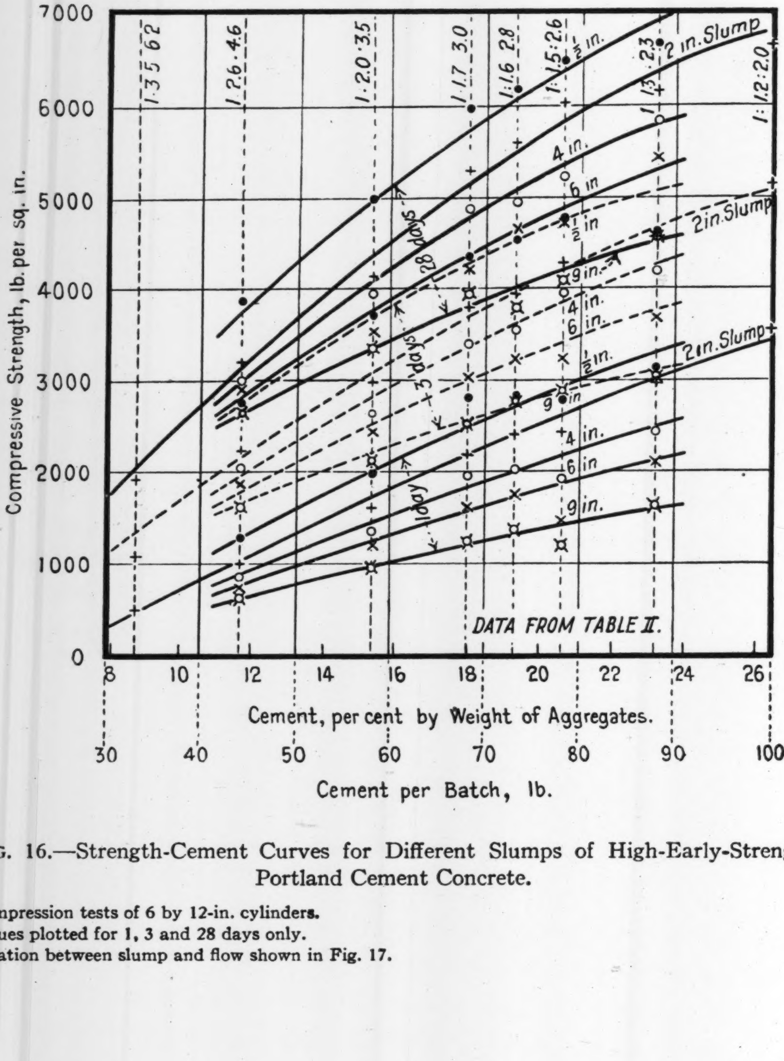

FIG. 16.—Strength-Cement Curves for Different Slumps of High-Early-Strength Portland Cement Concrete.

Compression tests of 6 by 12-in. cylinders.
Values plotted for 1, 3 and 28 days only.
Relation between slump and flow shown in Fig. 17.

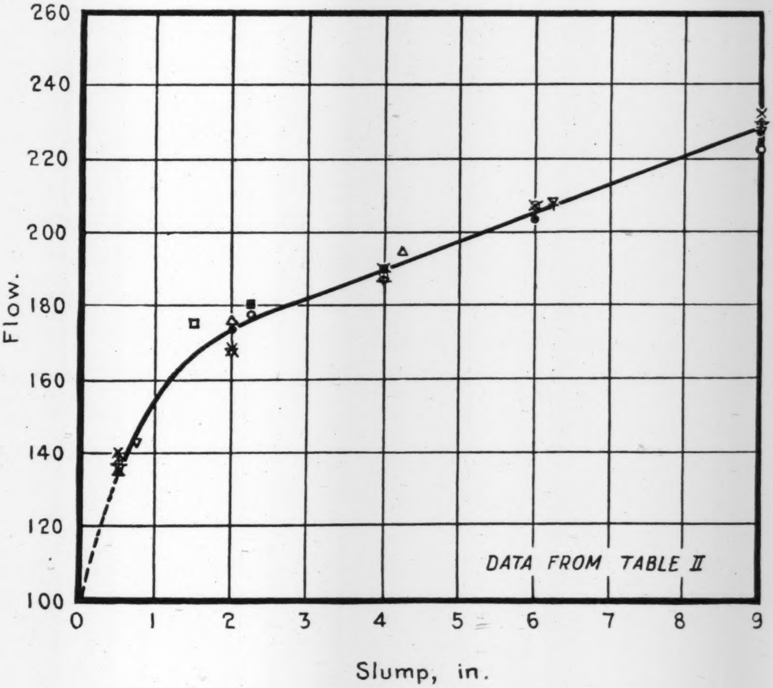

FIG. 17.—Flow - Slump Curves for Concrete Tests in Table II.

DISCUSSION

Mr. W. A. Slater[1] (*by letter*).—There are some conclusions in this paper which appear to place admixtures in an unfavorable light generally which do not seem to be warranted by the data presented.

All the concrete in which admixtures were used was of a rich mix (1:1.7:3.0) when compared with the concrete which goes into a large percentage of construction work other than road construction.

The author refers to his former paper[2] on the effect of hydrated lime as an admixture in concrete. In that investigation, as in the present one, flow is used as the measure of workability but in it a large range in richness of concrete was used as well as large range in flow. In the accompanying Figs. 1 and 2 strengths have been plotted as ordinates and the flow as abscissas for all the mixes of Table 27 of the appendix to the reprint of the former paper.[3]

An examination of these figures shows that (with the exception of three values) for the leanest mix, 1:6, the concrete with lime admixture gave higher strengths for a given flow (or greater flow for a given strength) than any of the concrete having no admixture. As the mix became richer the apparent advantage of the admixture gradually disappeared until for the richest mix (1:2) the concrete without admixture showed greater strengths than that with admixture for all but one case. The concrete reported in Table III of the author's present paper was of a 1:1.7:3.0 mix (a 1:4 mix when stated in terms of mixed aggregates), and it will be seen from the accompanying Fig. 1 that practically all of the advantage of the admixture had disappeared from the 1:4 mix. Thus it is apparent that in the present tests only mixes were used which had previously been shown to be better off without admixtures, and that with leaner mixes there probably would have been a gain in flow for a given strength or a gain in strength for a given flow due to the use of an admixture. To secure adequate information on the value of admixtures it seems to the writer that leaner mixes should have been included, and in their absence conclusions based on these tests should be so guarded that they could not by inference be extended so as to apply to leaner mixes.

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[1] Research Professor of Engineering Materials and Director, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa.

[2] D. A. Abrams, "Effect of Hydrated Lime and Other Powdered Admixtures in Concrete" *Proceedings*, Am. Soc. Testing Mats., Vol. XX, Part II, p. 149 (1920).

[3] Published as *Bulletin 8* of the Structural Materials Research Laboratory, Lewis Institute, Chicago.

It is not inferred here that the author proposes such an extension, but in the absence of an explicit statement that admixtures were not used in any lean mixes, it requires more careful reading than is usually given to such a paper to discover the omission of the leaner mixes and requires considerable familiarity with the entire field to discover the effect of such an omission on the general applicability of the conclusions.

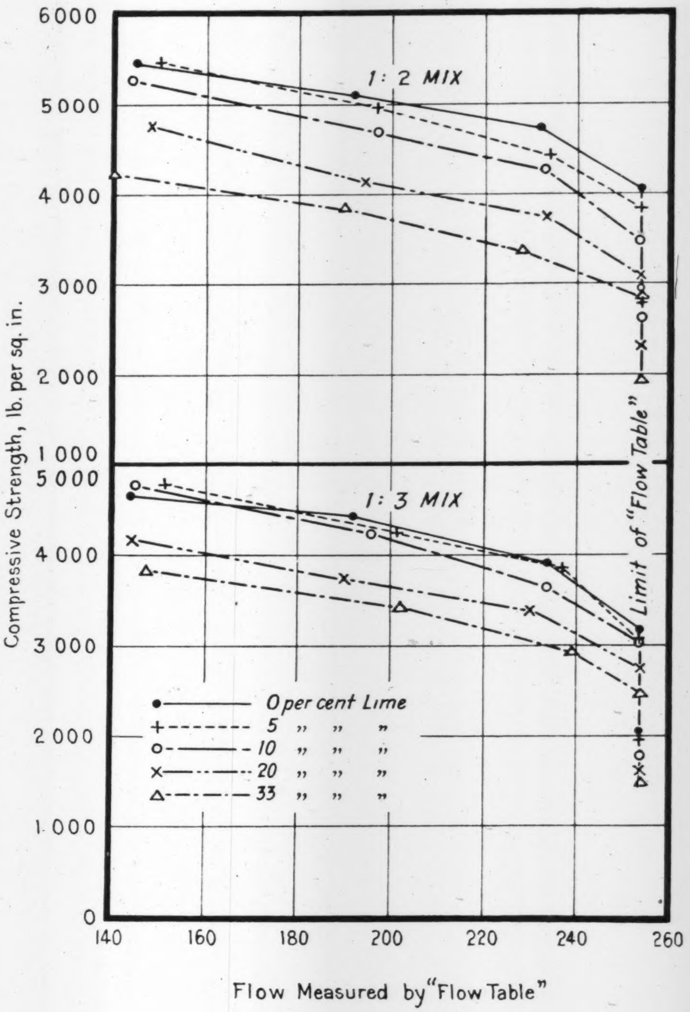

FIG. 1.—Strength-Flow Relations with Lime Admixture.

With no other measure of workability than flow, the writer does not believe that all the effects of various admixtures can be determined. "The consistent relation found between flow and water-cement ratio and flow and strength" is apparently the only evidence given in support of the conclusion that flow is "a satisfactory measure of workability." In view of the fact that, "This investigation has to do principally with studies of effect of admixtures on workability and

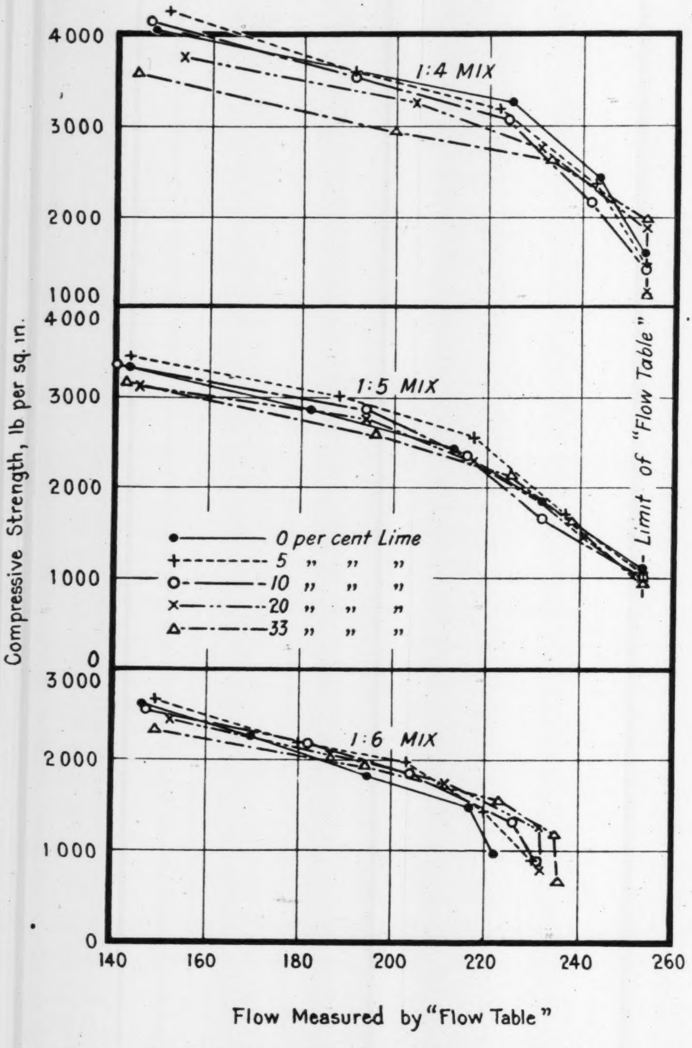

FIG. 2.—Strength-Flow Relations with Lime Admixture.

strength of concrete," the treatment seems to dismiss too lightly the results of serious research on methods of measuring workability. There are certain extremes of workability between which it is not difficult to select the most workable mix and any measure of workability which is accepted as satisfactory should give results which agree with one's judgment in the extreme cases. Pearson and Hitchcock[1] developed an apparatus for measuring workability and this apparatus was further studied and modified by Smith and Conahey.[2] The latter study indicated clearly that this modified workability apparatus gave results more in conformity with otherwise known facts regarding the workability than did the flow tests. Workability measurements with the apparatus developed by Pearson and Hitchcock or with that by Smith and Conahey would probably have been more difficult than those with the flow table, but it is disappointing to find no evidence that they were given any consideration.

The former paper compares workabilities of concretes to which an admixture had been *added*, while the present paper compares water-cement ratios of concretes of equal flow in which part of the cement had been replaced with an equal weight of admixture. Either method, of course, is permissible but since the need for an admixture is brought about partly by the lack of fine material (as indicated by the apparent advantage of admixtures in lean mixes, Fig. 1), it seems logical to make an addition of fine material rather than to substitute it for part of the cement. Figure 8 of the paper does not show how much the addition (not the substitution) of a given percentage of admixture affects workability, flow, water-ratio, or strength, nor is the information to be found elsewhere in the paper, but the former paper by the author (see the accompanying Figs. 1 and 2) shows that for leaner mixes than the one used here the addition of lime may increase the flow materially without decreasing the strength, in spite of the increased water-cement ratio. Whether the addition of an equal amount of cement would have improved the workability by an equal or greater amount may be open to question, but Pearson and Hitchcock conclude[3] that in a 1:2:4 mix the improvement in workability effected by a 12 per cent addition (by weight) of lime is about that which should be expected from a 25 per cent increase in the cement content. This is consistent also with the findings of Smith and Conahey[1] and is an additional reason for believing that the conclusion that "The workability of the concrete as measured by the flow test was not improved by any of the 14 powdered admixtures tested" would no longer hold if leaner mixes had been used, or if a more satisfactory measure of workability had been used than the water-cement ratio required for a given flow.

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[1] J. C. Pearson and F. A. Hitchcock, "A Penetration Test for the Workability of Concrete Mixtures with Particular Reference to the Effects of Certain Powdered Admixtures," *Proceedings*, Am. Soc. Testing Mats., Vol 23, Part II, p. 276 (1923).

[2] G. A. Smith and George Conahey, "A Study of Some Methods of Measuring Workability of Concrete," *Proceedings*, Am. Concrete Inst., Vol. XXIV, p. 24 (1928).

[3] J. C. Pearson and F. A. Hitchcock, "Economic Value of Admixtures," *Proceedings*, Am. Concrete Inst., Vol. XX, p. 326 (1924).

In conclusion the writer wishes to express his belief that if the proper weight be given to the cost of adding another material to concrete (even though that material itself be cheaper than cement), and the greater strength secured by using cement as the admixture, the advantages claimed by some for admixtures may entirely disappear, and in this he apparently does not disagree with the author. However, the paper under discussion does not add to the information already available on these points, and the acknowledged standing of the author tends to make his reversion to an inferior (relatively) criterion for workability a regrettable setback to progress toward solution of the workability problem.

MR. W. C. BRUCE[2] (*by letter*).—Mr. Abrams has reported only tests of admixtures in rich mixes (1:1.6:3.0 and 1:1.7:3.0), whereas in actual practice some of the important uses of admixtures are in medium and lean mixes. On page 625 of his paper the author makes the following statement: "A given increase in cement has a much greater effect in increasing strength of lean mixtures than the richer ones. The well-known law of diminishing return seems to apply to this case. This fact sets a distinct limitation on the economy of obtaining high strengths by means of rich mixtures." The same applies to the effect of admixtures, namely, admixtures have a greater effect on lean mixes than on rich mixes.

Therefore, the author trangresses his own ideas in the conclusions to his paper, wherein he attempts to extend the scope of the paper by applying his conclusions on rich mixes to concrete in general. Furthermore, in his previous article on admixtures,[3] Mr. Abrams states: "Rich concrete mixes showed a greater loss in strength due to powdered admixtures than the leaner ones." In this connection, it is of vital importance to remember that the mix used in testing admixtures has a pronounced effect on results and conclusions, not only regarding strength, but also regarding other important properties such as workability, durability, etc. For example, one of the salient features of tripoli as an admixture is that a medium mix plus the proper amount of tripoli as an admixture will have the workability, strength, water-tightness and durability of a rich mix, but without the disadvantage of the volume change which accompanies the use of rich mixes and which results in structural strains or cracks.

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[1] G. A. Smith and George Conahey, "A Study of Some Methods of Measuring Workability of Concrete," Fig. 12, *Proceedings*, Am. Concrete Inst., Vol. XXIV, p. 40 (1928).

[2] Admix Sales Manager and Chemical Engineer, American Tripoli Co., St. Louis, Mo.

[3] D. A. Abrams, "Effect of Hydrated Lime and Other Powdered Admixtures in Concrete," *Proceedings*, Am. Soc Testings Mats., Vol. XX, Part II, p. 149 (1920).

The inclusion of tests on eight different mixes of plain concrete gives an erroneous impression of the scope of the article regarding admixtures, because admixtures were tested in only one of these mixes.

In considering the author's conclusions, it should be remembered that tripoli is the only admixture which was not tested in more than one proportion, and that this proportion was a 10 per cent by weight replacement of cement, which is double the quantity recommended by the producers. In the case of tripoli, the author offers no experimental evidence to substantiate his conclusion that "The effect of admixtures was independent of the quantity of admixtures used." It should be pointed out that the producers of this material do not recommend it as a replacement for part of the cement, but specify that it should be used as an admixture in addition to the usual amount of cement for each mix. The results and conclusions would have been different if from 3 to 7 per cent of tripoli had been used as a replacement and vastly different if from 3 to 7 per cent had been used as an admixture as recommended.

Practically all the manufacturers of admixtures recommend that their materials be used as an admixture, and not as a replacement. Despite this, in these tests, admixtures were in all cases used to replace cement. Although no experimental evidence is given, Mr. Abrams states "the conclusions from these tests would have been exactly the same if the admixtures had been added to a constant quantity of cement." This is a contradiction of the author's statement in his previous paper on admixtures,[1] as follows: "The reduction in strength caused by replacing cement with an equal volume of hydrated lime was about 1¾ times that caused by adding hydrated lime."

Mr. Abrams states the computations for the water factor for the admixtures "take into account the fact that the admixture replaced an equal weight of cement." However, upon studying the computation, it is seen that it is merely a mathematical correction for the amount of cement replaced and does not take into consideration other factors, such as the point of maximum density, effect on colloidal and physical condition of the mix through changing the ratio of fines to the aggregate, etc. The fact seems to have been overlooked that there is more cementitious matter and fines per unit volume in "admixture" concrete, than there is in "replacement" concrete. In "admixture" concrete, the ratio of cement to aggregate is greater. In other words, the mix is richer in "admixture" concrete than it is in "replacement" concrete. Also in "admixture" concrete, the ratio of total amount of fine material (cement plus admixture) to amount of aggregate is greater than it is in "replacement" concrete. These differences cause differences in the properties of both the fresh and the finished concrete, and it is not possible to make mathematical corrections to accurately account for these differences. The only method to determine these differences with such a complex material as concrete is by direct experimentation. Because of proportions involved, these differences are not so pronounced in rich or in very lean mixes, but they are very pronounced in medium mixes. For example, when an admixture with a low water factor and high chemical purity is added (in amounts varying between 3 and 7 per cent by weight of cement) to any mix in the neighborhood of a 1:2:4 mix, the compressive and flexural strengths are markedly increased at all ages, even when the water factor of the admixture is allowed for.

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[1] D. A. Abrams, "Effect of Hydrated Lime and Other Powdered Admixtures in Concrete," *Proceedings*, Am. Soc Testing Mats., Vol. XX, Part II, p. 149 (1920).

Mr. Abrams' tests were made under ideal laboratory conditions and his condemnation of admixtures does not take into consideration the following factors met in the field, and which are beneficially influenced by the use of admixtures: (1) loss of cement by leakage through forms and into the ground where no forms are used, (2) uniformity, (3) difficulty in proper curing, especially in hot, dry climates, (4) watertightness, (5) durability of concrete as affected by its physical and chemical environment.

MR. G. A. SMITH[1] (*presented in written form*).—Mr. Abrams' paper has again brought to the attention of this Society a group of concrete materials which have received favorable recognition during the past few years. These materials are recommended and used as admixtures for the purpose of improving the workability of concrete. If the conclusions as set forth by Mr. Abrams are accepted without considering the methods employed in making the tests and without examining the test results reported, it would appear that admixtures as a whole might have no place in the field of concrete.

I feel, that in view of other tests made to determine the workability of concrete and in view of general field interpretation of this property, exception should be taken to the method used by Mr. Abrams for determining the effect of admixtures on the workability. Also, there develops from a study of the data presented certain trends for the mixes considered which would indicate that the results obtained for the basic mix used for determining the effect of admixtures on the strength of the concrete were not consistent with results obtained for other mixes tested. When comparisons are made taking into account this inconsistency it appears that the conclusions reached are not warranted.

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[1] Hydraulic Cement Engineer, Research and Development Dept., Johns-Manville Corp., Manville, N. J.

Any admixture which will be suitable and economical has two conditions that it must fulfill or it will not be satisfactory. First, it must improve the workability. Second, the concrete with which it is used must not give a lower strength than a similar plain mix gaged to the same consistency. Mr. Abrams has endeavored to show that not one of the materials used met these two requirements.

In arriving at the relative value of admixtures or of increased cement, Mr. Abrams used the slump test or the flow table as an indication of the degree of workability obtained or the relative workability of the various mixes. The design of a particular concrete mixture is based on three interrelated conditions: cost, strength and workability. Workability should characterize that property or condition of a concrete which permits of its being placed economically and in such a manner as will result in a satisfactory finished product. Workability can be brought about only by those items incorporated into the mix. The quantity and properties of the cement, the character, gradation and quantity of both fine and coarse aggregate, the amount of water within limits and the kind and amount of admixture used, each has its effect on workability.

For a given mix, everything else being constant, the amount of water used, provided the concrete is not too dry or soupy, is an index of the workability for that particular mix. In this case, the relative wetness or consistency is a function of the amount of water present and the slump test or the flow test, being tests for wetness or consistency, will indicate the relative workability of this one mix should the water be varied. On the other hand, should the mix be changed by varying the proportions, grading of the aggregates, or the use of an admixture, the same slump or flow may be obtained without giving any indication of a change in the inherent properties of the mix effecting the workability.

During the past two years or more, I have been interested in determining the relative merits of various admixtures. In the tests for determining the relative workability of a concrete mix or the workability index, the improved penetration apparatus was used. This apparatus has been described before this Society.[1] Without going into the details of making the tests, it should be pointed out that in all comparative tests both the sand and the river gravel were separated into different sizes and recombined by weight to give the same gradation at all times. All other conditions were made as nearly equal or comparable as possible.

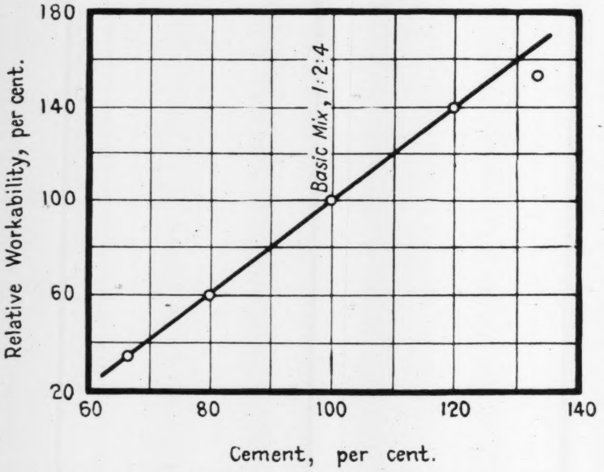

FIG. 3.—Change in Workability with Change in Mix.

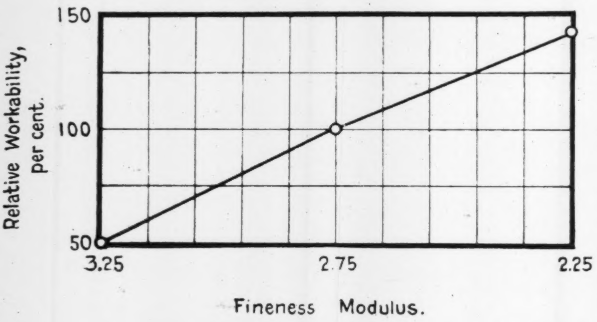

FIG. 4.—Change in Workability with Change in Fineness Modulus of Sand. Mix 1:2:4.

From data obtained from tests with the penetration apparatus, the accompanying Figs. 3 to 5 have been prepared. In Fig. 3 are shown results obtained when the cement content was varied from

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[1] G. A. Smith, "The Measurement of Workability of Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 505 (1928).

67 to 133 per cent by weight of the quantity used in a basic 1:2:4 by rodded volume mix. The consistency of each mix was measured on the flow table using fifteen $\frac{1}{8}$-in. drops. The flow is expressed as the percentage increase in the diameter of the specimen. In this figure it is seen that the relative workability varied almost directly with the quantity of cement used. Some time after the results shown in Fig. 3 were obtained, three separate groups of tests were made using 3 lb. of Celite per sack of cement (3.2 per cent by weight) and $33\frac{1}{3}$ per cent increase in cement as admixtures in the basic 1:2:4 mix. The improvement in workability given for each group is shown in the following table. The concretes in each group were gaged to the same flow.

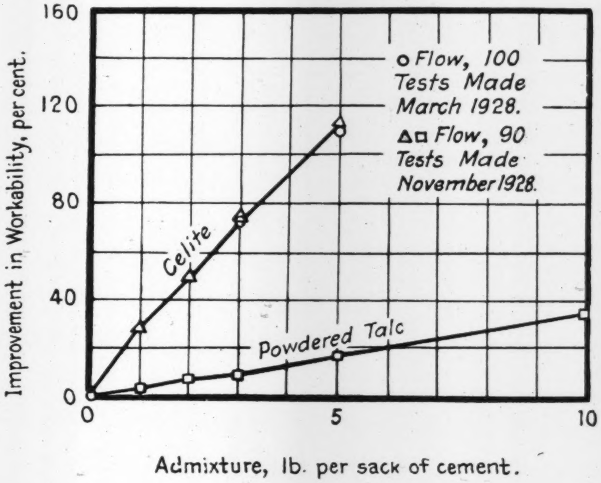

FIG. 5.—Change in Workability with Increase of Admixture.

ADMIXTURE	RELATIVE WORKABILITY, PER CENT OF BASIC MIX		
	GROUP I	GROUP II	GROUP III
None......................	100	100	100
Celite, 3.2, per cent..........	173	187	170
Cement, 33.3, per cent.......	175	176	169

It will be noted from the relative workabilities that Celite was about ten times as effective as cement in promoting workability. It will also be seen that although the concretes were gaged to the same consistency yet there was a marked difference between the basic mix, and the Celite and the additional cement mixes.

Figure 4 shows in a manner similar to Fig. 3 how the workability of the concrete varied with the fineness of the sand used. The concretes for these tests were proportioned by volume, but the actual weight of sand used was in proportion to 107, 106 and 102 or the weights per cubic foot for the 3.25, 2.75 and 2.25 fineness moduli respectively. All concretes were gaged to the same flow.

Figure 5 indicates the improvement in workability when different amounts of Celite and material of another class were added to the mix. As the quantity of the admixtures was increased the improvement in workability increased nearly in proportion to the amount of admixture used. It will be noted that results obtained with 3 and 5 lb. of Celite per sack of cement from tests made at different times indicated the same proportional improvement in workability. All comparable mixes were gaged to give the same flow.

The general conclusion that may be drawn from Figs. 3 to 5 is that although concrete mixtures may have the same consistency or flow, yet they also have other properties which are influenced by the proportions and the materials used and which are not indicated or measured by the slump test or the flow test. Results obtained with the penetration apparatus indicate changes in the properties of concrete which are parallel with recognized changes in workability due to changes in the mix. Although the flow test or slump test may be used to give indices of workability for different mixes taken separately as the water is varied, the indices are not comparable between mixes.

In considering the use of an admixture in concrete the commodity for consideration is improved workability. The question for the engineer, architect or the contractor to answer is, will it be worth the investment and will the designed mix with the admixture added give the desired strength? According to Mr. Abrams improved workability is obtained by increasing the water, or by using additional cement. He shows that by increasing the water in a given mix a greater flow is obtained. Within limits the greater flow is an index of better workability for a given mix. This is recognized so generally that the result is often disastrous so far as the strength of the concrete is concerned. The use of water as a workability agent over and above that called for in the design is coming to be looked upon as poor practice unless it is cared for by absorption by the aggregates or some other means.

According to Mr. Abrams the logical way to obtain improved workability is to use additional cement. It is generally recognized that of two mixes similar to those considered by Abrams gaged to the same consistency or flow the one containing the larger quantity of cement will be the more workable. Also that with the same water-cement ratio the richer mix is doubly more workable due to a larger cement content as well as an increase in flow. This larger flow factor is recognized by Mr. Abrams while the richness of the mix is not. On what, then, does the improvement in workability depend? If the workability of concrete gaged to the same flow were not a function of the cement content all concrete mixes having the same flow would be equally workable. This is most absurd, for it is almost universally recognized that a rich mix of a given slump is far more workable than a lean harsh mix at the same slump. In such mixes as were used in Mr. Abrams' tests, additional cement alone would not have resulted in improved workability.

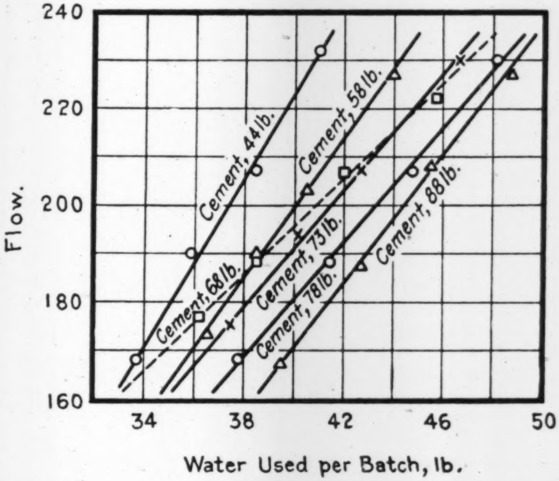

FIG. 6.—Change in Flow with Change in Water. Data from Table II.

In the accompanying Fig. 6 are plotted the flows and the corresponding quantities of water used in Mr. Abrams' tests (Table II) for the different mixes. It is seen that were no additional water used as the cement was increased the flow would be lowered materially. It is evident then, even to maintain the same flow with additional cement it is necessary to use additional water. If then the effects of the constituent materials other than the water are overlooked, as Mr. Abrams appears to do, his only criterion for increased workability is increased water—that is, water more than that required to maintain a given flow or slump.

There is little question but that, within limits, the workability generally improves with the addition of water or with an increase in the slump or the flow. This additional water, however, is injurious to the strength of the concrete unless some means is used to overcome its effect. This is one of the functions of an admixture or the additional cement. If the plasticising influence of an admixture or the extra cement used were nil, as Mr. Abrams indicates it to be, this would be the only function of the added materials. With the original added water resulting in better workability the use of a small amount of fine material such as cement or admixture will cause the mass to stiffen or will make it have the same apparent wetness as existed before either water or the admixture was added. Since the water is still in the mix, its effect on the workability can hardly be eliminated provided the same consistency is maintained as existed in the original mix.

Mr. Abrams speaks of the admixtures showing no improvement in flow although extra water was required to maintain a constant flow. This led to the conclusion that there was no improvement in workability. The same conclusion might be reached for additional cement. Were there no improvement due to the extra water and Celite, one would hardly expect that there would exist such consistent differences in the properties of the concrete as are indicated in Fig. 5.

It has been pointed out that the successful admixture must result in producing equal strength of the concrete. Mr. Abrams has attempted to show that the resulting strengths with admixtures were inferior to those obtained with the plain mixes. In considering the data he presented, a study was made with particular reference to the effect of diatomaceous earth on the strength as compared with the trends of all the results presented and not as compared with the results given for his basic mix. This mix was the 68 lb. of cement mix gaged with an amount of water equivalent to a water-cement ratio of 0.80. The resulting flow was 177. Had the flow been exactly 180, as was indicated for most of the concretes containing admixtures, somewhat more water would have been required and the strength would have been lower. This difference appears to be and would normally be a trivial matter but it led to the following analysis.

The accompanying Fig. 6 is similar to Fig. 8 of the paper except that flow is plotted against the actual quantity of water in the respective mixes instead of against the water-cement ratio. It will be observed that the one mix used in determining the effect of admixtures on the strength was the only one which did not show the same tendencies as were observed with mixes with smaller and larger cement contents. It certainly is not logical nor consistent that a mix containing 68 lb. of cement should give a higher flow with less water than was obtained for a mix containing 58 lb. of cement. From the intercepts of the curves in this figure and lines of equal flow, values were obtained which have been plotted to give the accompanying Fig. 7. This figure is similar to Fig. 9 of the paper. Based on the curve of 180 flow in Abrams' Fig. 9 the quantity of water that would be required with his basic mix to give a flow of 180 should have been 38.5 lb. This would correspond to a water-cement ratio of 0.853. From Fig. 7 it will be seen that 38.2 lb. would be required. This corresponds to a water-cement ratio of 0.846. Had the values for the basic mix and 73-lb. mix been omitted the line of 180 flow would have passed very near to the 38.5 lb. of water for the 68-lb. or basic mix.

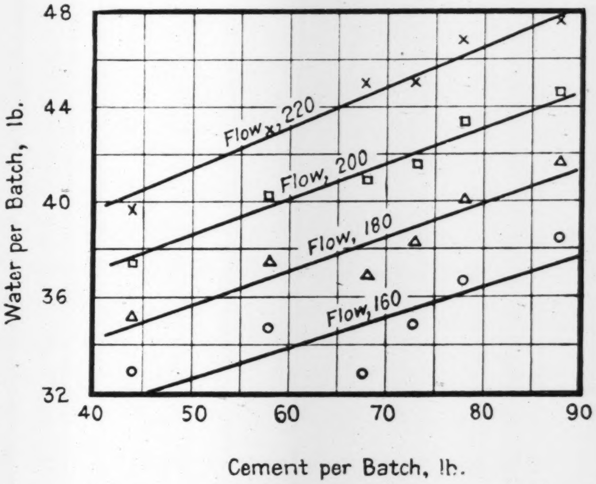

FIG. 7.—Water Required for Different Flows and Different Cement Contents. Data from Fig. 6.

From Figs. 6 and 7, therefore, it is quite evident that at least 38.2 lb. of water should have been used instead of 36.2 lb. This would have resulted in considerably lower strengths which would have been in line with those obtained in using 38.5 lb. of water or a water-cement ratio of 0.85.

In view of the fact that admixtures replaced an equal weight of cement a direct comparison of the effect on strength cannot be made without correcting the strength of the basic mix or those of the admixture concretes. Using the correction figures of 1.4, 1.2, 1.1, 1.0, and 0.9 per cent for each one per cent replacement of cement for the 1, 2, 3, 7 and 28-day strengths respectively given in Abrams' Table IV, the strengths of the mixes containing 1, 2 and 5 lb. of diatomaceous earth were corrected to correspond to the basic mix with the diatomaceous earth added. Strengths corresponding to a water-cement ratio of 0.846 were obtained from the curves shown in Abrams' Fig. 4. Strengths were also obtained by averaging all strengths (eight) in Table IV of Abrams' paper having water-cement ratios of 0.80 to 0.90. The corrected admixture concrete strength,

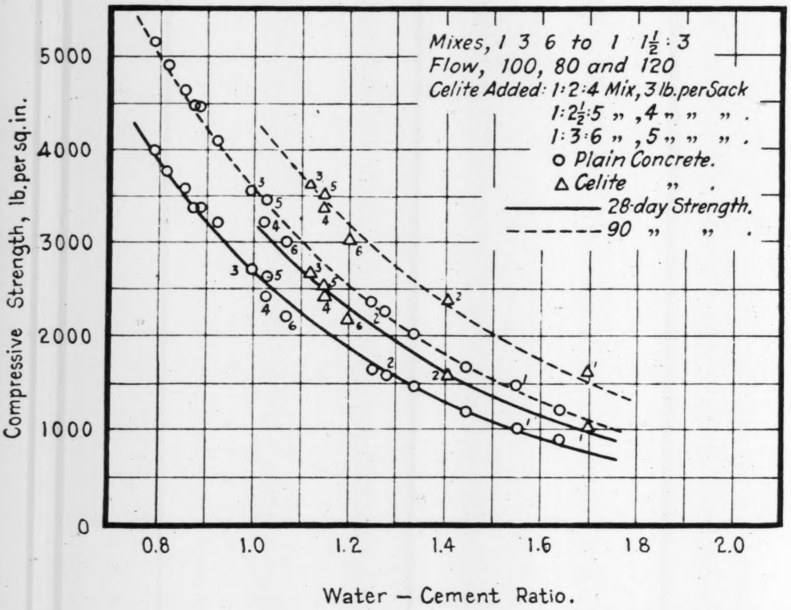

FIG. 8.—Water-Cement Ratio - Strength Relation of Plain and Celite Concrete.

the strength from the curves and the average strengths from the table of results are shown in the following table:

REFERENCE ABRAMS' PAPER	CEMENT, LB.	DIATOMACEOUS EARTH, PER CENT	WATER-CEMENT RATIO	COMPRESSIVE STRENGTH OF CONCRETE, LB. PER SQ. IN.				
				1 DAY	2 DAYS	3 DAYS	7 DAYS	28 DAYS
Fig. 4................	68	0	0.846	1950	3050	3500	4250	4950
Average Table II.....	68	0	0.838	1890	3030	3590	4280	4915
Table III corrected...	68	1.5	0.86	2026	3270	3583	4417	5371
Table III corrected...	68	2.9	0.90	1879	3076	3312	4369	5318
Table III corrected...	68	7.4	1.00	1617	2715	3350	3919	5082

From the strengths given in the table it is seen that the strengths for a water-cement ratio of 0.846 taken from the curves appear to be a little higher than would be expected from the average strength given by water-cement ratios between 0.80 and 0.90. However, comparing the strengths of admixture concrete with the strengths from Abrams' curves, it will be noted that the proportion of diatomaceous earth recommended for a mix as rich as the one used resulted in strengths if anything higher than did the plain mix. The next percentage of diatomaceous earth, nearly 50 per cent higher than is recommended, though giving strength slightly less than the plain mix at one and three days resulted in a higher average strength for all ages. Although the third proportion is so excessive as never to be used, it is of interest to note that the 28-day strength obtained was equal to that of the basic mix.

During the making of tests from which data shown in Figs. 3 and 4 were obtained concrete cylinder specimens were made from the mixes used. The cylinders were cured in a moist room at 70° F. and broken at 28 and 90 days. Results of the compression tests are shown in Fig. 8. Each point plotted is the average from the test of five specimens. It will be seen that the results for both the plain and the Celite concrete conform to the general water-cement ratio - strength relation in which the strength of the concrete is a function of the water-cement ratio. Comparable plain and Celite mixes were gaged to give the same flow. The point to be noted is that although additional water was used, when Celite was added to maintain the same flow, yet the same strength was obtained as was observed for the plain concrete.

Although the water-cement ratio was higher in mixes in which Celite was used when based on the actual quantity of water used yet the "effective" ratio or that part of the water not controlled by the Celite could not have been more since equal strengths were obtained. This would indicate that Abrams' water factor which he considered so detrimental to concrete strength, though it is additional to that used in the plain mix at the same consistency, plays no more part in determining the strength, when Celite is the admixture, than would water added to correct for absorption by the aggregates or if it were removed from the mix.

In a preceding paragraph it was mentioned that the question of cost was one that had to be answered by the person desiring an improvement in workability. There is no question but that the workability can be improved by the use of additional cement. That this is a fact is evidenced by the results presented in the earlier part of this discussion. But it is a question whether cement is the economical material to use for this purpose. It has been pointed out that the same degree of improvement in workability can be obtained with 3.2 per cent of Celite as can be had by using 33.3 per cent additional cement. That is, Celite is at least ten times as effective as cement in promoting workability. On the basis of $12 per ton for cement and $40 for Celite, it is seen that a given improvement in workability can be had by using Celite for one third to one fourth the cost of using additional cement.

Summing up it would appear that Abrams' conclusions based on the data presented are not warranted for the following reasons:

Where conditions of a mix other than the gaging water are changed neither the flow test nor the slump test is capable of denoting the relative workability of concrete mixes.

In comparing the effect of admixtures on the strength of concrete, strength results not at all characteristic with the general trend of results were taken as a basis. Using a reasonable and logical basic condition, diatomaceous earth when used in amounts which were not excessive and which were in accordance with the manufacturer's recommendations did not impair the strength.

The reduction in strength was not proportional to the water factor since the 1.5-per-cent addition of diatomaceous earth gave strengths consistently higher than were indicated by the general trend of all the data presented.

In general, using a fair interpretation of Abrams' data, it may be said that diatomaceous earth is not an "injurious adulterant" in concrete.

MR. C. P. DERLETH[1] (*by letter*).—Messrs. Slater, Smith and others have commented on certain limitations of the tests reported upon by Abrams. The writer wishes to offer the following comments in summary form:

1. Flow table tests were used as a measure of workability. The flow table is not a reliable measure of workability, particularly with the colloidal admixtures. Gelatinization, and its effect in eliminating segregation, a particular advantage to be gained from this type of admixture, is thus not taken into consideration. If additional cement is employed, as recommended, the use of additional water with the extra cement (keeping the same water-cement ratio) will provide additional so-called "paste," which results in additional workability and usually (in leaner mixes) a slight increase in strength. The use of an admixture in recommended proportions, together with the water absorbed or adsorbed by the admixture, yields additional lubricating paste which, per unit volume, is more effective in giving workability than an equal volume of relatively abrasive additional segregating paste from cement and water. Workability is affected by variations in the ratio of combined fine and coarse aggregates to paste.

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[1] Colloy Products Co., St. Louis, Mo.

From Fig. 17 it is apparent that slump and flow are very closely related and that the flow table tests should accordingly be recognized as a measure of consistency. Neither is recognized as a measure of workability.

2. Very rich mixes only were used. Table I covers 1:4 concrete; Table III includes 1:1.7:3.0 concrete. It is well known that an admixture is more effective in the leaner mixes both as regards strength and workability. Since improved placing equipment enables the satisfactory handling and placing of leaner mixes the modern trend is toward increasing the amount of coarse aggregate for economic and durability reasons. Most of the tests, including all of Series 18A, were carried out using high-early-strength portland cement yielding in the rich mixes extremely workable plain concrete, largely eliminating the beneficial effects of the admixtures on size grain, gradation and distribution of the coarser ground portland cements.

3. A weight of cement equal to the weight of admixtures used was omitted in the admixture concrete. To give comparable results, there should be added to the admixture-concrete strength ratios, percentages equalling the percentage reduction in amount of cement used. When thus interpreted, a number of admixtures, particularly at 7 and 28 days or later ages show increases in strength over plain concrete, notwithstanding the additional water used in the admixture concrete and the adverse conditions of the tests.

4. Additional water was used in all admixture concretes, necessary in the drier low-slump concretes to maintain equal flow or slump, though not workability. It is generally recognized that admixture concretes of lesser slump can be as easily placed as plain concretes of somewhat greater slump. Further, in the wetter consistencies, there is usually available as so-called "excess water," sufficient water to satisfy the absorption requirements of the usual weights of admixtures used, and therefore generally no additional water is required in these wetter mixes to maintain the same slump and because of the tendency of the excess water to cause segregation, which ill effect is largely offset by the admixture, additional workability is provided, also slight increase in yield.

Abrams' broad conclusion that "all of the commercial admixtures tested must be classed as injurious adulterants due to their high water factors" does not coincide with conclusion 10 of his previous paper[1] as quoted below:

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[1] D. A. Abrams, "Effect of Hydrated Lime and Other Powdered Admixtures in Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. XX, Part II, p. 149 (1920), reprinted with Appendix as *Bulletin 8* of the Structural Materials Research Laboratory, Lewis Institute, Chicago.

> "Hydrated lime and other powdered admixtures used in these tests slightly increased the workability of the leaner mixes (1:9 and 1:6) as measured by the slump test. Ordinary mixes (1:5 and 1:4) were little affected; richer mixes (1:3 and 1:2) were made less plastic."

Conclusion "*h*" of the appendix to Abrams' previous paper is also of interest:

> "With a constant quantity of mixing water in 1:4 and 1:5 mixes (and of course variable flow) hydrated lime showed an increase in strength of 0.61 per cent for each 1 per cent of lime."

This conclusion is confirmed by Conahey's findings[1] relative to the absorption by an admixture having the effect of reducing the water available for water-cement ratio computation.

A total of 4 cylinders, one for each mix, using colloidal clay (Bentonite), one cylinder only using a highly siliceous colloidal admixture in the "standard" portland cement series, were crushed at 3 months' age (see Table I). No tests on diatomaceous earth at ages greater than 28 days were reported. Data on this limited number of cylinders do not justify the implied conclusion that, with the exception of diatomaceous earth, serious loss in strength at all ages results from the use of commercial admixtures. However, the paper and discussions are helpful in bringing out that the paste derived from an admixture and its absorbed water is productive of workability and increased yield. The latter factor while influencing economics was not brought out by Abrams but should be considered.

It is regretted that Abrams did not include important detailed data on the strengths, flow, water-cement ratio, etc., of specimens made from the 10 batches of plain concrete, since these (averaged) were used as the standard for comparison. It is further not understood why, if the water content could be so closely controlled in the admixture concretes as to yield a flow of exactly 180 in 36 of a total of 44 batches, the variation of the remaining 8 batches in every case being on the wet side (with corresponding lower strength), the average of the 10 batches of plain concrete used as the standard for comparison should average a flow of only 177, insufficient water having been used (with corresponding greater strength). If additional water had been used in the plain concrete to obtain a flow of 180, the strength comparison results quite probably would have shown less, if any, reduction and in many instances increases in strength over that of the plain concrete.

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[1] George Conahey, "The Effect of Admixtures on the Water-Cement Ratio-Strength Relation of Concrete," Appendix VIII to Report of Committee C-9 on Concrete and Concrete Aggregates, *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 373 (1928).

In all tables and figures the admixture designated by Abrams as colloidal clay (D) is Colloy, a colloidal silica admixture, comparable in chemical analysis to diatomaceous silica, though quite different in physical characteristics. This particular admixture cannot by chemical or petrographical analysis be classified as a typical clay, colloidal or otherwise, and should not be so classified as Abrams has done.

The broad conclusion that all colloidal admixtures are injurious is unwarranted. (See Series 18A, Table III, Reference No. 70, Colloidal Clay (D).) This may be seen when a 3-per-cent addition is made to the compressive strengths to compensate for the 3-per-cent reduction in quantity of cement due to the substitution of 2 lb. of Colloy for 2 lb. of cement, and similarly compensating the admixture strength ratios by 3 per cent as shown in the following comparison:

	Quantities of Materials								Compressive Strength, lb. per sq. in.					Strength Ratio				
		Admixture			Water													
Reference Number	Cement, lb.	Kind	Per Cent	Pounds	Pounds	Water-Cement Ratio	Flow Value	Slump, in.	1 Day	2 Days	3 Days	7 Days	28 Days	1 Day	2 Days	3 Days	7 Days	28 Days
39	68	None	...	..	36.2	0.80	177	2.25	2190	3220	3800	4440	5280	100	100	100	100	100
70	68[a]	Colloidal Silica Colloy. (D)	2.9	2	38.0	0.87	180	2.75	1785[a]	3310[a]	3845[a]	4655[a]	5250[a]	82[a]	100[a]	101[a]	105[a]	100[a]

[a] Corrected value.

Considering that not more than 12 lb. of colloidal silica is recommended by the manufacturer per cubic yard in 1:2:4 or richer mixes, the amount 2.9 per cent employed by Abrams is excessive, and correspondingly the other mixes using 5.0, 7.4 and 14.7 per cent of Colloy are greatly beyond the usual practice and should not be included in any consideration of the subject.

The per dollar yield of paste, used without increasing or decreasing the effective strength of the in-place concrete, also, the value of the resulting possible increases in yield from the added pastes with or without additional aggregates, are not covered in Abrams' paper but deserve investigation. In *Bulletin 8,* referred to above, Abrams in conclusion 14 stated that "Powdered admixtures increased the yield of concrete."

That standard portland cement does not produce satisfactory workability is acknowledged in that, admixture cements are being marketed by some manufactures of cement and others are producing more finely-milled and more workable, high-early-strength cements. The value of an admixture is not measurable in compressive strength cylinders which represent perfectly puddled concrete; their true function is to improve workability. Recognizing that admixtures are effective in improving workability, their effect in reducing strength variations, honeycombing, and in reducing the hazard of the labor element justifies their use, accounting for the widespread use of admixtures in important work.

MR. DUFF A. ABRAMS[1] (*author's closure by letter*).—The principal criticisms of my paper have been based on:

1. Use of flow table as a measure of workability of concrete;
2. Replacing cement by an equal weight of admixture;
3. Use of rich mixtures;
4. Use of larger quantities of admixtures than recommended by the manufacturers.

These general criticisms will be discussed first, then further consideration will be given to other points raised in the discussions.

Use of Flow Table.—While it may be admitted that the flow table is not a perfect measure of workability for all types of concrete it should be pointed out that the mixtures included in my paper were not unusual but fall within the range of workable mixtures as measured by any method. The flow table is not satisfactory for extremely lean mixtures, dry mixtures, wet mixtures or those deficient in fine material. However, it will be noted that none of these limitations apply to the concrete used.

In further justification of the use of the flow table, it may be pointed out that this apparatus is in use by practically all of the laboratories in this country which make any pretention of scientific control of the concrete tested. Reports which have come to my attention during the past few weeks show that the flow table is in use also by the leading concrete testing laboratories in the following countries: Austria, Belgium, Brazil, Cuba, Denmark, France, Germany, Japan, Sweden, Switzerland.

The need for a more satisfactory measure of workability is generally recognized; however, the difficulty of the problem is illustrated by the fact that at least four of the outstanding American research laboratories in this field have devoted considerable attention during the past five or six years to this problem and have not yet reached a satisfactory solution. In fact, the flow table is now being used as a measure of control in all of these laboratories pending results of further research.

Replacing Cement.—The same criticism was made of an earlier series of tests as a result of which further parallel tests were carried out using hydrated lime both as an admixture and as replacing cement.[1] The results showed that exactly the same conclusions would be reached by both methods. Additional reason for replacing the cement by admixture in my tests was that we, as cement manufacturers, have been urged to introduce these materials at the mill. Obviously, this would necessitate replacing equal weight of cement.

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[1] Director of Research, International Cement Corp., New York City.

Use of Rich Mixture.—A number of the men discussing my paper have made a considerable point of the richness of the concrete in which the admixtures were used. Whether a mix is rich or not seems to depend entirely on the field in which we are working. The mixtures tested were those commonly employed in concrete roads and are frequently used in buildings and many other types of concrete work. The tendency during the past few years has been in the direction of richer mixtures for most types of work.

Use of Larger Quantities of Admixture.—In most instances a wide range of percentages of admixtures was used in order to determine the effect of any intermediate percentage. My observation leads me to believe that manufacturers have no monopoly on knowledge concerning this subject; many of them seem to have no basis whatever for the recommendations made, and make frequent changes in their recommendations.

Mr. Slater states that "the paper under discussion does not add to the information already available on these points . . ." Since others may have reached the same conclusion it may be well to point out some of the new features of this paper.

1. Certain admixtures were included which have not heretofore been subjected to study of this kind.

2. The tests of concrete were made at earlier periods than have been employed.

3. Part of the tests were made with a high-early-strength cement of unusual fineness.

Certain other new features of the report may be of interest.

1. The development of the water-factor as a method of determining the water requirements for concrete is a new approach to the whole question of concrete mixtures so far as the inter-relation of portions of materials and workability are concerned.

2. It is pointed out that increasing the quantity of cement gives a diminishing return in concrete strength. This conclusion shows the impracticability of predetermining the 28-day strength of concrete

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[1] See "Effect of Hydrated Lime and Other Powdered Admixtures in Concrete," *Bulletin 8*, Structural Materials Research Laboratory, Lewis Institute, Chicago. The appendix published in the *Bulletin* gave the results of tests made subsequent to the publication of the original paper, *Proceedings*, Am. Soc. Testing Mats., Vol. XX, Part II, p. 149 (1920).

from a knowledge of the 7-day strength as proposed by Mr. Slater. A formula of this kind will give strengths which are too low for lean mixtures, too high for rich mixtures and for the high-early-strength cements.

3. In the note under Fig. 4, I have shown that the constant A in the formula appears to be independent of the age of the concrete.

4. The inter-relation of the quantities of water, cement, strength and flow given in Fig. 9 are presented in this form for the first time.

Any of the foregoing items would have been considered a major contribution to our knowledge of concrete had they appeared in separate papers rather than in a report on admixtures.

While Mr. Slater objects to the methods used in my tests, it is noted that he ends his discussion by admitting that he does not disagree with my conclusions.

Mr. Bruce states that ". . . one of the salient features of tripoli as an admixture is that a medium mix plus the proper amount of tripoli as an admixture will have the effect of workability, strength, water-tightness and durability of a rich mix . . ." This is the usual salesman statement, but none of these claims are borne out by any investigations which have come to my attention. We had no reason to believe that this admixture possessed any unique properties and the tests made indicate that it does not. Most of the concrete tests reported by the admixture manufacturers omit either the quantity of mixing water or the workability, or both, and indicate that no attention was given to these factors. Such tests can have little standing in a serious discussion of this subject.

Mr. Smith calls attention to the water-cement ratio of the concrete mix which was used as a basis for comparison of the effect of admixtures. A further study of the original tests on which Reference No. 39, Table II, is based indicates that the actual quantity of water used was somewhat higher than that given in the table due to the fact that the aggregates in some of the tests were not in a room-dry condition. This would fully account for the apparent discrepancies to which attention is called. The strength and flow would not be changed, but the water-cement ratio should be slightly higher than that given.

The indiscriminate use of powdered admixtures in concrete is being prohibited by many engineers. An instance recently came under my observation in which a powdered admixture was used in a concrete street pavement. Portions of this pavement showed distinct surface cracks where the admixture was used; the cracks did not appear in the following day's work when the admixture was discontinued. The same pavement contained a number of holes on the surface about the size of a teacup. The contractor explained that the holes were caused by balls of the admixture which were not broken up in the mixer.

The following quotation from a proposed specification submitted to the American Concrete Institute by Committee 502, by A. R. Lord, Author-Chairman, shows the attitude of an engineer of long and varied experience in a wide range of concrete work.

> Nothing shall be added to the essential ingredients of concrete (portland cement, fine and coarse aggregate, and water) without the approval of the architect or engineer in writing and then only such materials shall be added as have been thoroughly tested by reputable independent investigators so as to demonstrate their effect on the strength, elastic properties, permeability and permanence of concrete produced from materials such as are being used on this work.[1]

Mr. Lord's comment on this section of the specification may be of interest:

> Admixture manufacturers and salesmen will not enthuse over Article 10. We have meager reliable information on most admixtures—practically nothing but salesmen's loose talk on others. The instructions on the printed label are sometimes absurd. Any product that has been on the market as long as this ought to be less of a secret-formula patent medicine and more of a staple article. The tests that are known are none too encouraging and many large corporations which have made tests for their own information and have decided to dispense with most admixtures as a result, are unwilling to publish their data for the benefit of the rest of us. I feel that this specification contains the only sensible provision that can be made in the present situation. If generally accepted it should lead to the accumulation of adequate and reliable test data to supplement the widely variable data of experience now generally circulated—the favorable opinions being circulated by salesmen and the unfavorable by other salesmen.[1]

In any consideration of admixtures in concrete we should ask ourselves, what is a reasonable expectation of its behavior? There must be some property of the admixture itself which can be determined in advance by measurement, which would explain its behavior in concrete. The "water-factor" as given in my paper and the "normal consistency" of the materials seem to offer the desired explanation. The admixture manufacturers seem to have little comment to make on these properties of their materials.

A considerable increase in the water requirement for concrete cannot be accompanied by an increased strength. The effect of increasing water on strength, water-tightness and durability is so well recognized that it requires little further discussion.

The only conclusion which can be reached from our present knowledge is that portland cement does not require a "crutch" in the form of an inert powdered admixture in order that we may make a high-grade concrete to meet any requirement.

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[1] *Journal*, Am. Concrete Inst., November, 1929.

MODULUS OF ELASTICITY AND POISSON'S RATIO FOR CONCRETE, AND THE INFLUENCE OF AGE AND OTHER FACTORS UPON THESE VALUES

By Raymond E. Davis[1] and G. E. Troxell[2]

Synopsis

The paper describes long-time laboratory tests made on plain concrete cylinders to determine the axial and lateral deformations produced by axial compressive stresses. Following the customary practice, the ratio of unit compressive stress to the corresponding total unit axial deformation is designated as the secant modulus of elasticity even though the deformations are not instantaneous in character and are not altogether elastic. The relation between lateral and axial strains accompanying compressive stresses is given by values of Poisson's ratio.

The tests were made to establish the manner in which the modulus of elasticity and Poisson's ratio are influenced by (1) richness of mix, (2) age of concrete, and (3) repeated loads. Supplementary tests were made to determine the influence upon the modulus of elasticity of (4) moisture content of the concrete, and (5) length of period of sustained load. The paper presents the results of these tests and discusses the findings. Among other things, it is shown that the stress-strain ratio or modulus of elasticity is less for a concrete when dry than when wet, under a sustained load is greatest immediately upon application of load and continues to decrease for a long period of time, is not necessarily highest for the richest mix and strongest concrete, and increases quite rapidly with the age of the concrete for the first year but after three years the increase with time is small. Poisson's ratio is greater for low stresses than for higher ones, it increases somewhat with age up to about 1½ years, after which it remains nearly constant, and does not seem to be influenced by strength of concrete or by richness of mix.

Introduction

When a material is subjected to compressive stress, deformations are produced within the mass and there occurs a maximum decrease in length in the direction of stress and a maximum increase in length in a plane at right angles thereto. The former is called the axial deformation, and the latter the lateral deformation. The ratio of total unit stress to total unit axial strain is designated as the secant modulus of elasticity and the ratio of the unit lateral to unit axial deformation is called Poisson's ratio.

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[2] Assistant Professor of Civil Engineering, University of California, Berkeley, Calif.

For most of our structural materials these stress-strain relations are not constant. This is particularly true of concrete for which values of these ratios are generally understood to vary with magnitude and period of application of stress, with age and quality of concrete, and with moisture and temperature conditions. Furthermore, concrete is not perfectly elastic even at low stresses, since it fails to resume completely its original dimensions upon release of stress, and continues slowly to deform over a long period of time under the action of sustained stress. Hence, in so far as concrete is concerned, the term modulus of elasticity is somewhat of a misnomer, since the stress-strain ratio includes plastic as well as elastic deformations.

In connection with the early work of the Arch Dam Investigation of the Engineering Foundation it appeared that existing information regarding the behavior of concrete under stress, for many of the conditions occurring in practice, was quite inadequate. Accordingly, in 1925 there were begun in the Materials Testing Laboratory of the University of California extensive tests to determine the elastic and plastic properties of concrete under axial compressive stress. Though certain of the tests are still in progress, the results to date are here presented with the thought that they may be of interest and value to those having to do with the design and construction of concrete structures in general.

In carrying out the experimental work, more than ordinary attention was given to securing measurements of high precision and to maintaining constant conditions of storage. For the principal tests axial and lateral strains were measured to millionths of an inch and many of the specimens, including those subjected to sustained stress, were stored under constant conditions as regards temperature and humidity.

Scope of Tests:

For convenience of discussion the tests may be divided into four series: Series 1, begun in 1925 and extending over a period of 3 years, to determine the influence of age and richness of mix upon modulus of elasticity and Poisson's ratio; Series 2, begun in 1926 and still in progress, to determine the effect of age and repeated applications of stress upon the modulus of elasticity and Poisson's ratio; Series 3, extending through the year 1928, to determine the influence of moisture content upon the modulus of elasticity; and Series 4, begun in 1926 and still in progress, to determine the effect of constant sustained stress upon the stress-strain ratio, here called the sustained modulus of elasticity.

The results are based upon observations on more than 350 specimens. The specimens for the first three series just mentioned (numbering about 300) were 6 by 12-in. cylinders; those for the fourth series (numbering more than 50) were 4 by 14-in. cylinders.

For Series 1, tests were made on three concretes, varying only as regards richness of mix, at several ages up to three years, to determine the axial and lateral deformations taking place immediately upon application of load. That is, the load was applied in increments, in the customary manner, and after each increment the deformations were observed, each specimen being tested to failure without removal of load. The aggregates were beach sand and crushed metamorphic sandstone. The storage was in damp sand.

For Series 2, tests were made upon a single grade of concrete at various ages, to determine the axial and lateral deformations taking place immediately upon application of load, some of the specimens being loaded to failure, and some of them unloaded when a stress of 1000 lb. per sq. in. was developed, to be similarly tested at a later age. That is, at a given age one group of specimens not previously tested were loaded progressively to failure, axial and lateral deformations being observed after each increment of load; at the same age a second group not previously loaded and a third group previously loaded were tested in the same manner up to 1000 lb. per sq. in. The aggregate was composed entirely of crushed granite, and the storage was in damp sand.

For Series 3, tests were made at two ages upon three concretes, varying as to richness of mix, to determine the axial deformations taking place immediately upon application of load, half of the specimens of each mix being water soaked at time of test and the remainder being air dry. Each test was carried to failure without removal of the load, and the deformation was observed after each increment of load was applied. The aggregate was gravel. The storage was under water, except that the air-dry specimens were stored in warm, dry air for a week before testing.

For Series 4, each specimen was loaded at a given age and thereafter the stress was constantly sustained. The specimens are composed of granite concrete of the same mix used for Series 2. Among specimens, loading conditions were varied as to age at which the load was applied and the magnitude of the sustained stress. For the several conditions of loading, some of the specimens have been stored in water and the remainder in air at constant humidity. Upon application of load and periodically since that time, axial deformations were measured. The tests have now been in progress for more than two years without the load having been removed. The deformation taking place subsequent to the application of load is referred to as the flow and the tests are called flow tests.

Apparatus and Equipment:

For the tests of Series 1 and 2 axial and lateral deformations were determined by use of the apparatus illustrated in Fig. 1. The longitudinal extensometer consists of two diametrically opposite axial side-bars (one of which is shown in the figure) clamped to the specimen by means of rubber bands. At the lower end of each side-bar is a knife-edge which bears against the specimen. The upper end of each bar bears against a lozenge through which passes a rod with mirror mounted on one end. As the specimen deforms, the lozenge, and hence the mirror, is rotated. The deflection of a beam of light

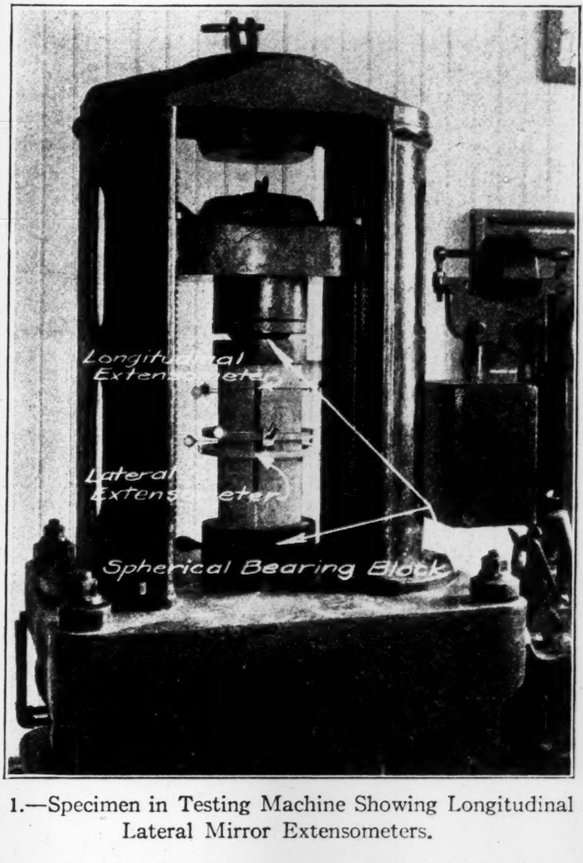

FIG. 1.—Specimen in Testing Machine Showing Longitudinal and Lateral Mirror Extensometers.

reflected by the mirror is measured by the apparent movement of cross-hairs over a scale as observed through a telescope.

The instrument for measuring the lateral deformations consists of two aluminum collars separated by two small rollers with mirrors mounted thereon. The rollers and collars are held in contact by means of springs. A point on the upper collar is in contact with the specimen diametrically opposite to a similar contact point on the lower collar, so that an increase in the diameter of the specimen produces a relative lateral displacement of the two collars, and hence produces rotation of the rollers on which are mounted the mirrors. The angle of rotation of the mirrors is determined by the deviation of a beam of light upon a scale as observed with a telescope. This apparatus is fully described by A. N. Johnson.[1] Unit deformations to millionths of an inch may be readily measured with this device.

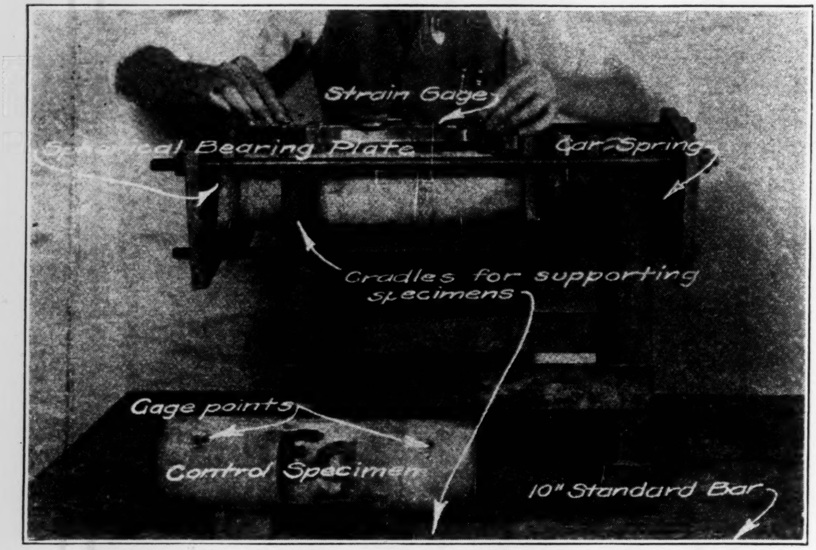

FIG. 2.—Showing Specimen with Strain Gage in Position of Taking a Measurement.

For Series 3 axial deformations were measured directly by means of a compressometer equipped with a Last Word dial, which indicates double the deformation in an 8-in. gage length. The device consists of two collars, one of which is clamped to the specimen at the third points of the circumference, and the other of which is clamped to the specimen at diametrically opposite points. To the collars are attached two diametrically opposite longitudinal bars, one of the bars being fixed to the collars at either end, and the other bar being fixed to one of the collars at one end and bearing against the lever of the dial fixed to the other collar at the other end. By means of this instrument unit deformations were determined to hundred-thousandths of an inch.

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[1] A. N. Johnson, "Direct Measurement of Poisson's Ratio for Concrete," *Proceedings*, Am. Soc Testing Mats., Vol. 24, Part II, p. 1024 (1924).

For Series 4 the axial deformations were measured by means of a 10-in. strain gage of the fulcrum-plate type, illustrated in Fig. 2, similar to that developed in connection with the Arch Dam Investigation of Engineering Foundation. At the third points of the circumference of each specimen stainless steel gage plugs were inserted. Deformations were measured to hundred-thousandths of an inch.

In order to eliminate as many variables as possible the specimens of Series 4 were stored in specially constructed rooms where the temperature was maintained constantly at 70° F. Those in dry storage were in an atmosphere of which the relative humidity was maintained constantly at 70 per cent.

In Fig. 2 is also shown the apparatus by means of which the load on each of the specimens of Series 4 was maintained at a constant value. This apparatus consists of a system of rods and plates and a heavy car spring which reacts against one end of the test cylinder. The desired load was applied through the car spring by means of a compression testing machine especially designed for this purpose. When the required stress was reached, nuts at the ends of the rods were tightened to a firm bearing and the testing machine load was released. After the testing machine load was released the compression in the car spring, and hence the load on the specimen, was maintained by tension in the rods. The changes in length of the rods due to stress, and changes in length in the specimen after the load was first applied, were so small as compared with the deformation of the spring that the stress in the specimen was maintained practically constant.

EFFECT OF AGE AND RICHNESS OF MIX UPON MODULUS OF ELASTICITY AND POISSON'S RATIO (SERIES 1)

The concrete for Series 1 was of three mixes as follows: 1 part o cement to $3\frac{1}{2}$ parts of mixed aggregate; 1 part of cement to $4\frac{1}{2}$ parts of mixed aggregate; and 1 part of cement to 6 parts of mixed aggregate, all measurements by volume. Measured by weight the aggregate was composed of 35 per cent of Monterey sand (a fine beach sand all passing the No. 14 and retained on the No. 100 sieve); 25 per cent of metamorphic sandstone screenings varying from No. 8 to $\frac{5}{8}$-in. sieve sizes; and 40 per cent of $\frac{1}{2}$ to $1\frac{1}{2}$-in. crushed metamorphic sandstone. The sieve analysis curve approached Fuller's ideal curve for maximum density. Santa Cruz cement was used. All specimens were mixed to the same consistency as measured by a slump of approximately 1 in.

The specimens, 216 in number, were all 6 by 12-in. cylinders, capped with neat cement. After 24 hours the specimens were stored in damp sand until just prior to testing, when the ages varied from 40 to 1035 days. In order to insure uniform bearing stress the ends of the specimens were bedded in a very thin layer of plaster of Paris at the time of test, and each specimen was tested with a spherical bearing block at either end.

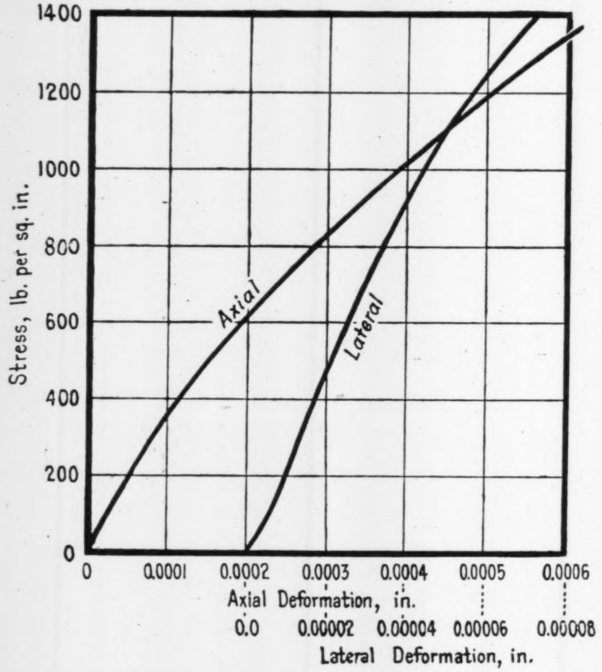

FIG. 3.—Typical Stress-Strain Curves for a Single Specimen at the Age of Three Months.

The mirror extensometers, previously described and shown in Fig. 1, were observed simultaneously by two observers using two transit telescopes and two illuminated scales. The testing machine was operated slowly by hand in order to eliminate the possibility of vibrations affecting the extensometers, and care was taken to maintain practically constant temperature conditions by testing at night without artificially heating the laboratory room, and by storing the specimens near the testing machine for several hours prior to testing.

Both axial and lateral stress-strain measurements thus determined gave remarkably smooth curves passing through the origin. The curves in general show that the axial strain relation is practically a straight line for low stresses, and the indications are that the richer the mix and the older the concrete, the straighter the axial stress-strain curve becomes. But regardless of age or mix the curve in every case becomes concave downwards at the higher stresses.

The lateral stress-strain curves are of the same shape as the axial stress-strain curves, except for the lower stresses near the origin where the former curves are frequently slightly concave upward.

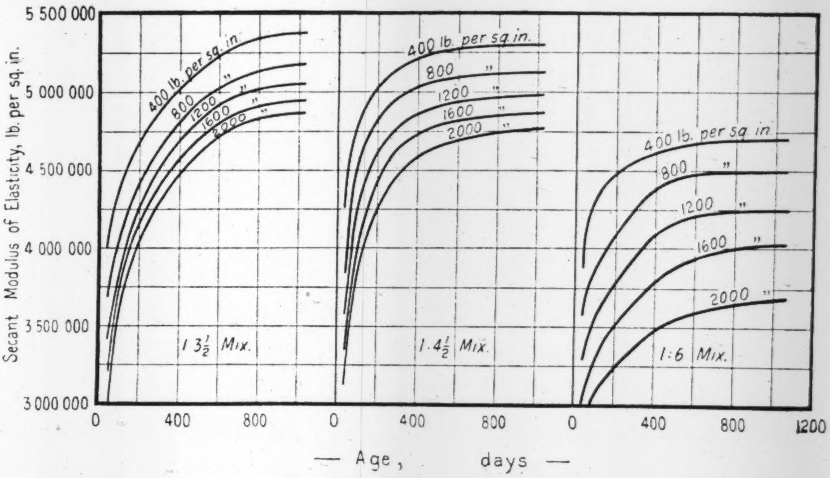

FIG. 4.—Effect of Age upon Secant Modulus of Elasticity—Sandstone Concrete.

Figure 3 shows typical stress-strain curves for a single specimen at the age of 3 months.

Modulus of Elasticity:

The effect of age upon the modulus of elasticity is shown by Fig. 4, where for each of the three mixes of Series 1 are given curves showing the relation between age and the secant modulus for various unit stresses from 400 to 2000 lb. per sq. in. A study of the curves shows that the modulus increases rapidly during the first six months or so. At the age of 3 years the indications were that for all mixes the modulus was still increasing, though the rate of increase, particularly for the lean mix, was small. It will be observed that the medium-mix secant modulus for the lower stresses and later ages exceeds 5,000,000 lb. per sq. in. Comparing the three groups of curves of the figure it is

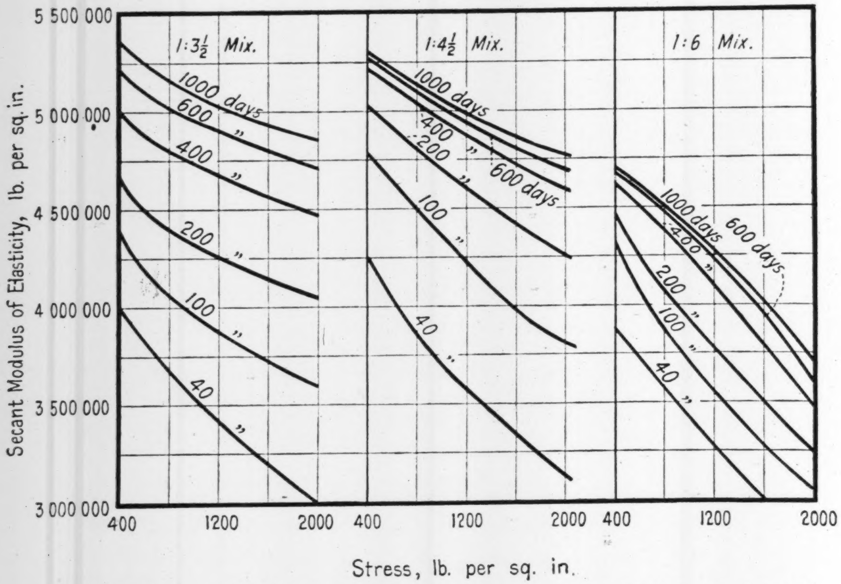

FIG. 5.—Relation of Unit Stress to Secant Modulus of Elasticity—Sandstone Concrete.

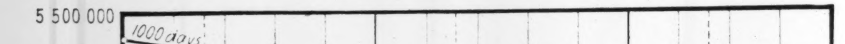

FIG. 6.—Effect of Richness of Mix upon Secant Modulus of Elasticity—Sandstone Concrete.

seen that the richer the mix, the greater the relative increase in the value of the modulus with time. Thus for the stress of 800 lb. per sq. in. the increase in the modulus between the ages of 200 and 1000 days is for the 1:3½ mix from 4,400,000 to 5,200,000 lb. per sq. in., and for the 1:4½ mix from 4,800,000 to 5,100,000 lb. per sq. in.

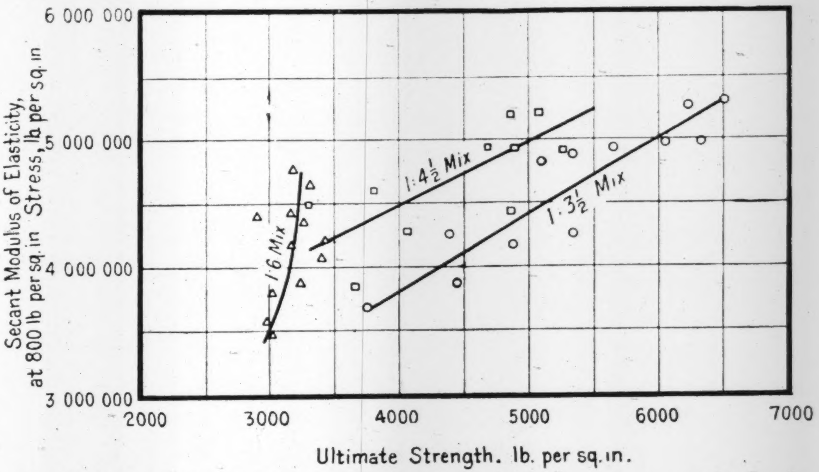

FIG. 7.—Relation Between Ultimate Strength and Secant Modulus at 800 lb. per sq. in.—Sandstone Concrete.

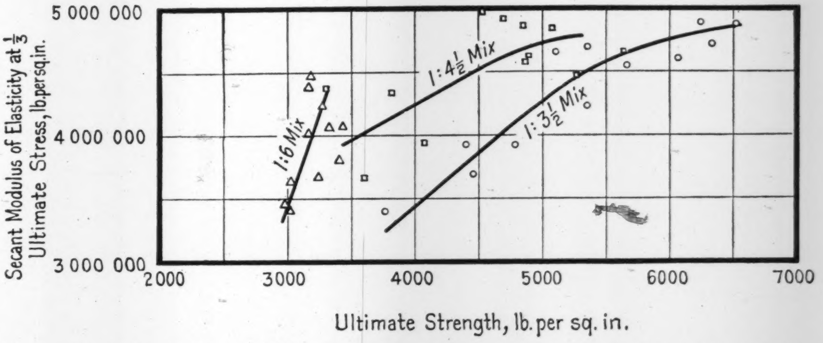

FIG. 8.—Relation Between Ultimate Strength and Secant Modulus at One-Third of Ultimate Strength—Sandstone Concrete.

The relation between secant modulus and unit stress is shown by the curves of Fig. 5 where for the several ages, from 40 to 1000 days, values of the modulus are given for the three mixes of Series 1. It will be noted that for the lean mix the rate of change of modulus with stress is about the same at one age as at another, as is indicated by the general parallelism of the several curves of the 1:6 mix. But for the richer mixes the decrease of modulus with increase in stress is distinctly less at the later ages than at the early ages.

The effect of richness of mix upon the modulus of elasticity is indicated by Fig. 6 in which are shown, for three unit stresses, the secant modulus at various ages for the three mixes of Series 1. It is instructive to observe that regardless of stress up to the age of about 600 days the modulus is greater for the 1:4½ mix than for the 1:3½ mix, but at greater ages the modulus for the 1:3½ mix becomes the larger.

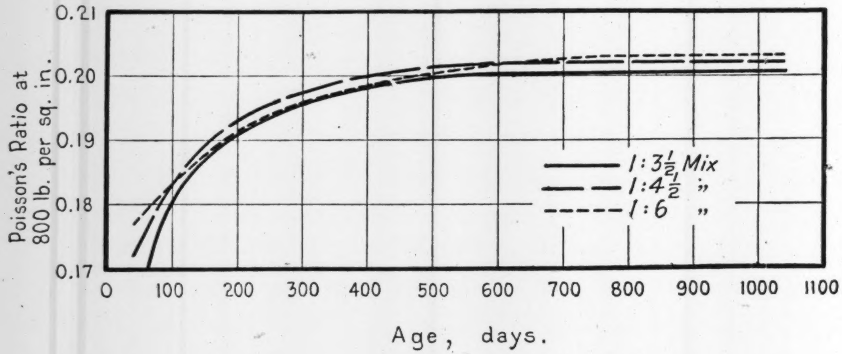

FIG. 9.—Effect of Age on Poisson's Ratio—Sandstone Concrete.

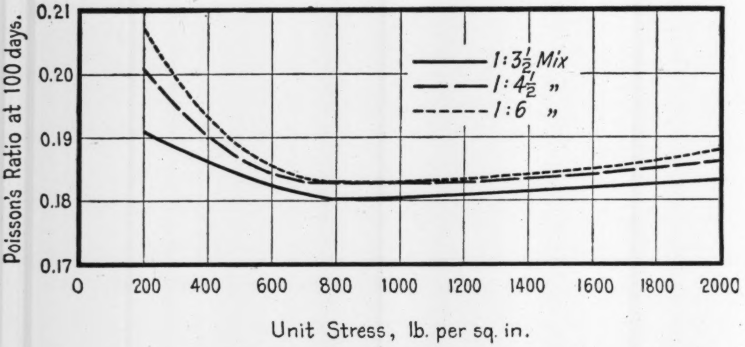

FIG. 10.—Variation of Poisson's Ratio with Stress—Sandstone Concrete.

In order to establish the general relationship between secant modulus of elasticity and ultimate strength, the mean curves of Figs. 7 and 8 were drawn, each plotted point representing the average values of modulus and strength as determined by tests on six specimens. For Fig. 7 the ordinates are values of the modulus at 800 lb. per sq. in. and for Fig. 8 the ordinates are values of the modulus at one-third of the ultimate strength. It is seen that in these figures the points for each mix fall into a more or less well defined group, and that the positions of the curves are in the order of their cement ratios. That is, there is a fairly well defined relation between modulus of elasticity and ultimate strength, but this relationship is not the same for rich mixes as for lean ones. The diagrams for the 1:6 mix show that while during the period over which the tests extended the modulus increased considerably, the ultimate strength increased but slightly; hence, it may be concluded that for leaner mixes the ultimate strength is not a definite indication of the value of the modulus. But for the richer mixes the ultimate strength increases rapidly with increase in modulus.

Poisson's Ratio:

While the tests of Series 1 demonstrate that values of Poisson's ratio are influenced by age, unit stress, and richness of mix, the variations of this ratio are relatively small as compared with corresponding variations of the modulus of elasticity with similar changes in conditions.

The effect of age upon Poisson's ratio is given by Fig. 9, where for each mix there is plotted a graph showing the relation between age and Poisson's ratio at 800 lb. per sq. in. It is seen that the ratio is only slightly influenced by richness of mix but that it does increase with age, its value being about 0.17 at 40 days and about 0.20 at 16 months. Poisson's ratio appears not materially to increase after the age of 2 years.

The relation between Poisson's ratio and unit stress is shown by Fig. 10, where for each of the three mixes are given values of the ratio at the age of 100 days for various unit stresses from 200 to 2000 lb. per sq. in. It is seen that within the range of normal working stresses Poisson's ratio decreases somewhat as the stress increases, lying between the values of 0.21 and 0.18. At the higher stresses the ratio appears to become more nearly constant. The figure makes it apparent that the richness of mix has little effect upon Poisson's ratio regardless of the magnitude of stress.

EFFECT OF AGE AND PREVIOUS LOADING UPON MODULUS OF ELASTICITY AND POISSON'S RATIO (SERIES 2)

The concrete for Series 2 was of a single mix composed of 1 part of Colton portland cement to 5 parts by weight of crushed granite graded from fine to coarse, all passing the 1½-in. sieve and 5 per cent passing the No. 100 sieve. The water-cement ratio was 1.0 and the average slump was 3.2 in.

The specimens were 6 by 12-in. cylinders capped with cement mortar. The molds were removed at 24 hours and thereafter the storage was in damp sand at 70° F.

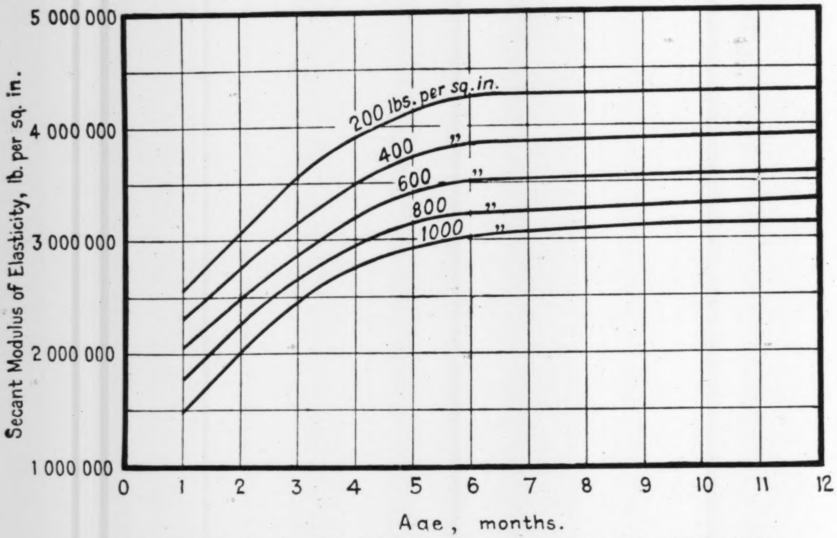

FIG. 11.—Effects of Age of Granite Concrete upon Modulus of Elasticity.

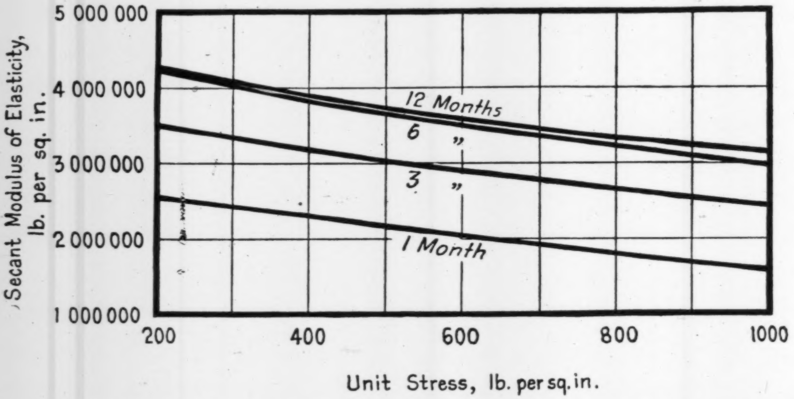

FIG. 12.—Variation of Secant Modulus of Elasticity with Intensity of Stress—Granite Concrete.

Both axial and lateral deformations were observed with mirror extensometers in the same manner as for the specimens of Series 1. Some of the cylinders were tested to failure at the ages of 28 days, 3 months, 6 months, and 1 year, and the remainder were stressed at these ages to 1000 lb. per sq. in. when the load was removed and the specimens were again stored in sand, the loads later to be repeated.

The age and manner of testing the seven groups into which the series was divided are given in Table I. Thus it is seen that group No. 6 was first tested to 1000 lb. per sq. in. at the age of 3 months, with similar tests made at 6 months and 1 year, while group No. 8 was first tested at the age of 1 year.

TABLE I.—AGES AND CONDITIONS OF LOADING, GRANITE CONCRETE.

Group	Age of First Test	Maximum Stress, lb. per sq. in.	Ages at Which Load was Reapplied
No. 1	28 days	Ultimate	
No. 2	3 months	Ultimate	
No. 3	6 months	Ultimate	
No. 4	1 year	Ultimate	
No. 6	3 months	1000	6 months; 1 year
No. 7	6 months	1000	1 year
No. 8	1 year	1000	

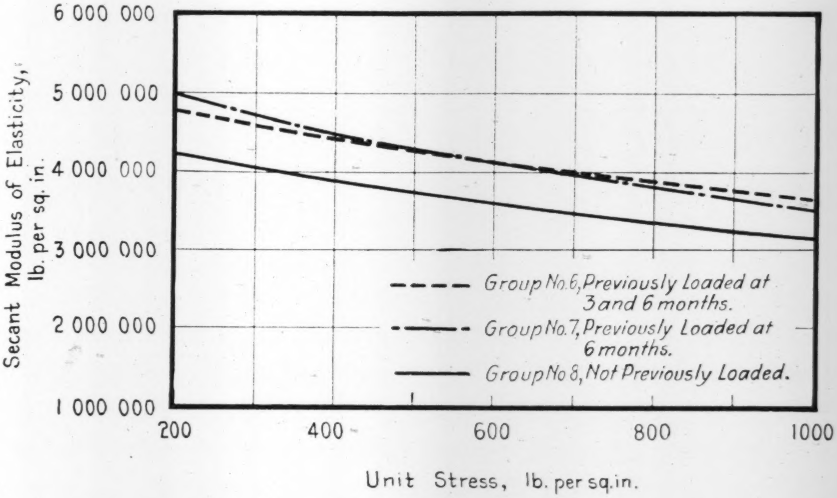

FIG. 13.—Effect of Previous Loading on Secant Modulus of Elasticity.

Modulus of Elasticity:

The effect of age upon the modulus of elasticity of the granite concrete of Series 2, when the load is not repeated, is shown in Fig. 11, where for various unit stresses the secant modulus is plotted against age. From the curves it appears that the modulus increases quite rapidly during the first 6 months, but only very gradually thereafter.

These results are somewhat in contrast to those for sandstone concrete (see Fig. 4) for which the value of the modulus continued rapidly to increase after the age of 6 months. Also the modulus of the granite at a given age and stress is considerably lower than that of a corresponding sandstone concrete at the same age and stress.

The relation between unit stress and secant modulus at various ages is shown in Fig. 12, from which it appears that within the ages one month to one year, and below a stress of 1000 lb. per sq. in., the modulus decreases almost in a direct ratio as the stress increases. The figure also makes it clear that the rate of change of the modulus with change in stress is about the same for one age as for another, as is indicated by the close parallelism of the several curves. The close agreement between the moduli at 6 months and 1 year will be noted.

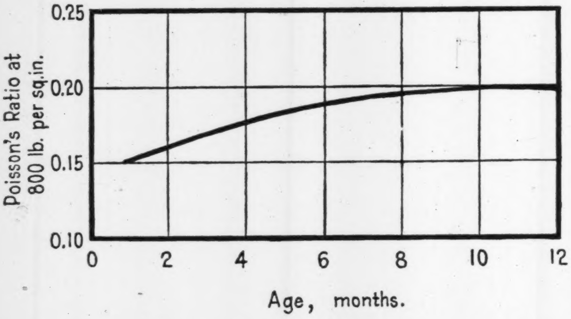

FIG. 14.—Effect of Age upon Poisson's Ratio—Granite Concrete.

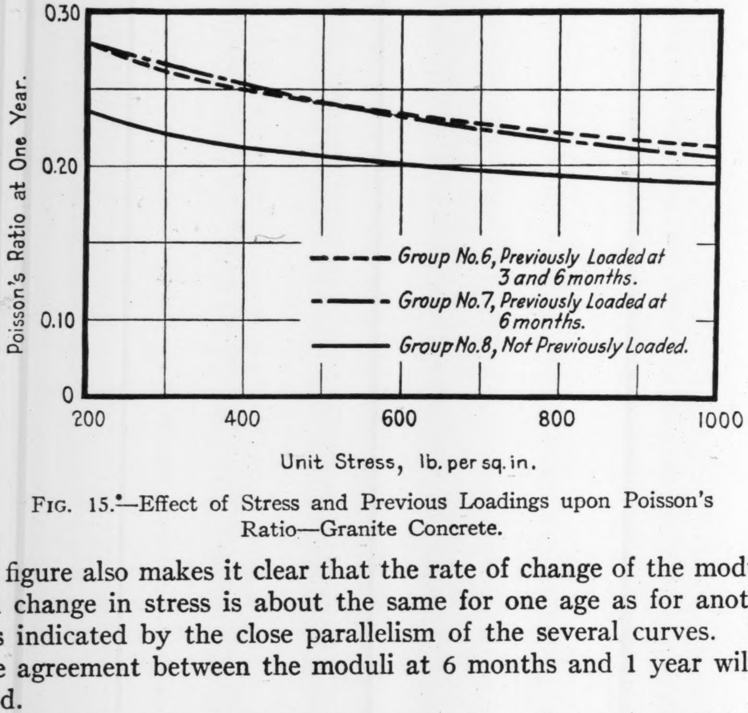

FIG. 15.—Effect of Stress and Previous Loadings upon Poisson's Ratio—Granite Concrete.

The effect of previous loadings upon the modulus of elasticity at the age of one year is shown in Fig. 13. It is seen that group No. 6

previously stressed to 1000 lb. per sq. in. at ages of 3 and 6 months, and group No. 7 previously stressed to 1000 lb. per sq. in. at the age of 6 months, both exhibit higher moduli than does group No. 8 which had not been loaded prior to the application of the stress of 1000 lb. per sq. in. at 1 year. But there is no marked difference in the behavior of groups Nos. 6 and 7, their curves nearly coinciding for much of these lengths.

Poisson's Ratio, Granite Concrete:

The influence of age upon Poisson's ratio is shown in Fig. 14, in which this ratio for a stress of 800 lb. per sq. in. is plotted against age in months. Poisson's ratio is seen to increase from 0.15 at 1 month to nearly 0.20 at 1 year, which is substantially the same as for the sandstone concretes of Series 1 (see Fig. 9).

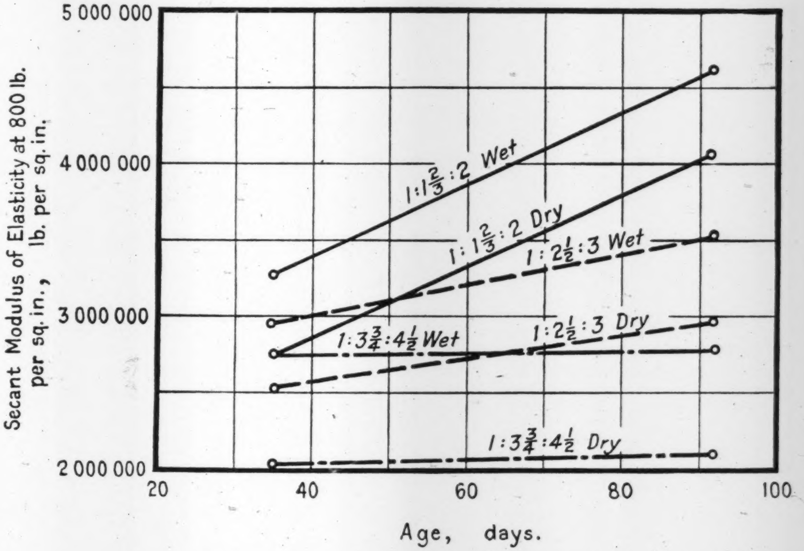

FIG. 16.—Effect of Moisture Content on Modulus of Elasticity.

The results of Series 1 and 2 indicate that Poisson's ratio increases with age but is little influenced by richness of mix or by character of aggregate within the limits of the tests.

The effect of magnitude of stress is shown in Fig. 15 for granite concrete at the age of 1 year, from which it appears that Poisson's ratio decreases as the stress increases, the rate of decrease being more rapid at the lower stresses than at the higher ones. Thus the values of Poisson's ratio for the specimens not previously loaded (group No. 8) were 0.235, 0.200 and 0.185 at the respective stresses of 200, 600, and 1000 lb. per sq. in.

Figure 15 also shows the effect of previous loadings upon the value of Poisson's ratio, the two upper curves being for groups Nos. 6 and 7 which were previously stressed to 1000 lb. per sq. in., the former group at ages of 3 and 6 months and the latter group at the age of 6 months only. It appears that the effect of earlier loadings has been to produce substantial increases in the magnitude of the ratio. Thus for both group No. 6 and group No. 7 values of Poisson's ratio were 0.280, 0.235, and 0.210 at the respective stresses of 200, 600, and 1000 lb. per sq. in.

A comparison of Figs. 15 and 13 indicates the striking similarity between the effect of magnitude of stress upon Poisson's ratio and upon modulus of elasticity.

EFFECT OF MOISTURE CONTENT UPON MODULUS OF ELASTICITY (SERIES 3)

The concrete for Series 3 was of three mixes as follows:

Mix	Mix, parts by weight			Water-Cement Ratio
	Cement	0 to $\frac{1}{4}$-in. Sand	$\frac{1}{4}$ to $1\frac{1}{4}$-in. Gravel	
No. 1	1	$1\frac{2}{3}$	2	0.54
No. 2	1	$2\frac{1}{2}$	3	0.76
No. 3	1	$3\frac{3}{4}$	$4\frac{1}{2}$	0.96

The aggregate was a Coast Range gravel from Niles, Calif., screened into the two sizes indicated above. The combined aggregate had a fineness modulus of 5.5. The mixes were all of the same consistency as measured by a flow of 150. The water-cement ratios shown in the above table were absolute values, due allowance being made for moisture in the aggregates.

The specimens were stored under water at 70° F. after the molds were removed at the age of 24 hours. Seven days prior to making the tests, one-half the specimens were removed from water storage and were placed in dry air at 130° F. They were removed from this storage and allowed to come to normal air temperature a few hours before testing. The remaining specimens were removed from water storage a few hours before crushing in order that their surfaces might be dry at time of test.

The tests of both wet and dry specimens were made at the ages of 35 and 142 days, loads being carried to the ultimate and deformations for increments of load being measured with the Last-Word compressometer previously described.

The results of the tests are shown in Fig. 16, in which values of the secant modulus of elasticity at 800 lb. per sq. in are plotted against age, for both wet and dry specimens. It is seen that, regardless of mix or age, the wet specimens exhibited substantially higher moduli than did the corresponding dry specimens, the difference amounting to from 450,000 to 750,000 lb. per sq. in., depending upon the mix. It is significant that this difference seems to be nearly independent of the age, as is indicated by the parallelism of the corresponding graphs for wet and dry groups of the three mixes.

At this point it is of interest to observe that while the modulus of elasticity was 12 to 30 per cent higher for wet concrete than for dry, the percentage increase being greater the leaner the mix, the ultimate strength exhibited by the specimens was higher for the dry concrete than for the wet by about the same ratios. That is, for a given concrete the effect of free moisture within the mass is to *decrease* the strength and to *increase* the modulus of elasticity.

EFFECT OF SUSTAINED STRESS UPON SUSTAINED MODULUS OF ELASTICITY (SERIES 4)

The concrete for Series 4 was composed of 1 part of portland cement to 5 parts of crushed granite graded from fine to coarse, all passing the 1½-in. sieve and 5 per cent passing the No. 100 sieve. The concrete was identical in composition with that for the specimens of Series 2, previously described.

The specimens were 4 by 14-in. cylinders. They were removed from the molds at 24 hours, when those to be loaded at ages of 2 and 7 days were stored in water and those to be loaded at later ages were stored in damp sand until the time of loading.

The specimens which were placed under load may be divided into fifteen groups. After loading, nine of these groups were stored under water at 70° F., and the remaining six groups were stored in air at 70° F. and 70 per cent relative humidity. Among the groups, conditions vary as regards age at which load was applied and intensity of sustained stress. The ages of loading were 2, 7, and 28 days, and 3 months; and the stresses applied were 200, 300, 600, 900, and 1200 lb. per sq. in.

Just prior to the loading of each specimen, strain-gage measurements were taken along each of the three axial gage lines at the third points of the circumference of the cylinder; and similar observations were taken as soon as the load was applied. The difference between these two observations is here termed the *instantaneous deformation*. Each specimen, clamped in its load sustaining apparatus, was then

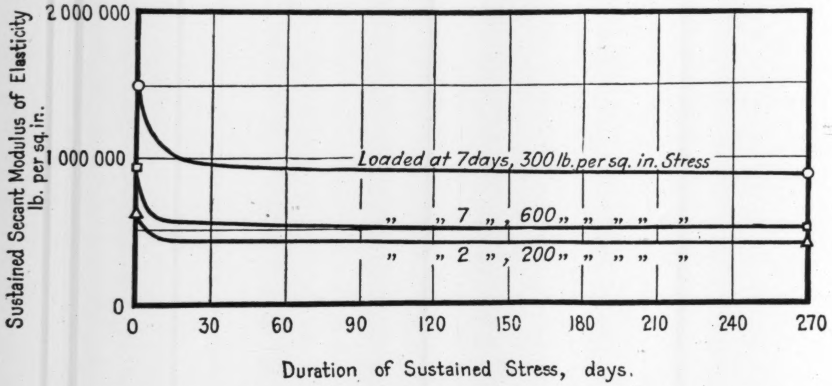

FIG. 17.—Effect of Sustained Stress on the Sustained Secant Modulus of Elasticity Loads Applied at Ages of Two and Seven Days.

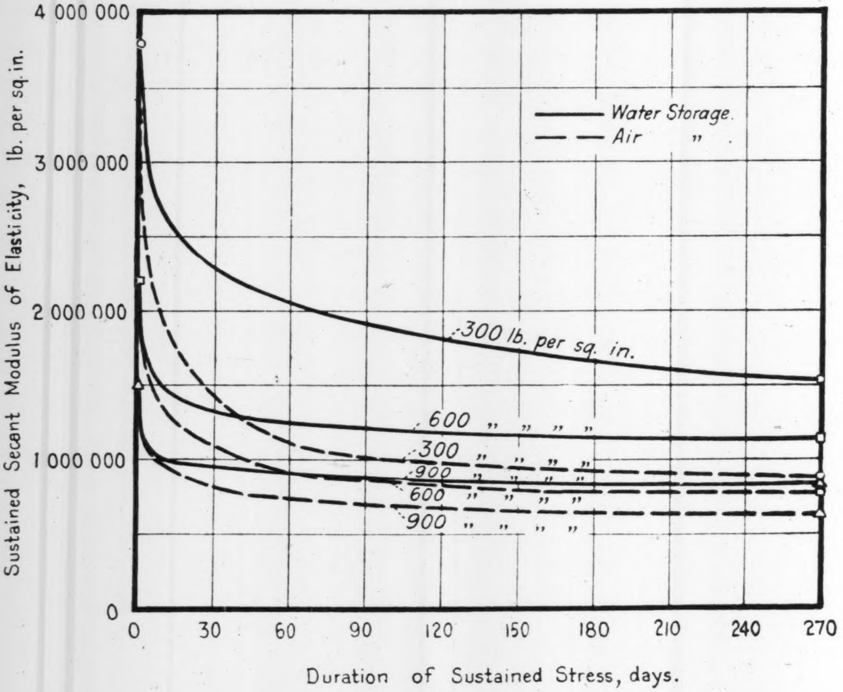

FIG. 18.—The Effect of Sustained Stress upon the Sustained Secant Modulus of Elasticity, Stress Applied at 28 Days.

placed in the desired storage (water at 70° F. or air at 70° F. and 70 per cent humidity) and strain-gage observations were taken periodically. The deformation taking place subsequent to the application of load, allowance being made for volume changes in the concrete due to causes other than stress, is here termed the *flow.*

In order to make proper allowance for changes in the length of loaded specimens due to causes other than stress, unloaded control specimens for each age of loading and condition of storage were observed for changes in length at the same time that measurements of the corresponding loaded specimens were taken.

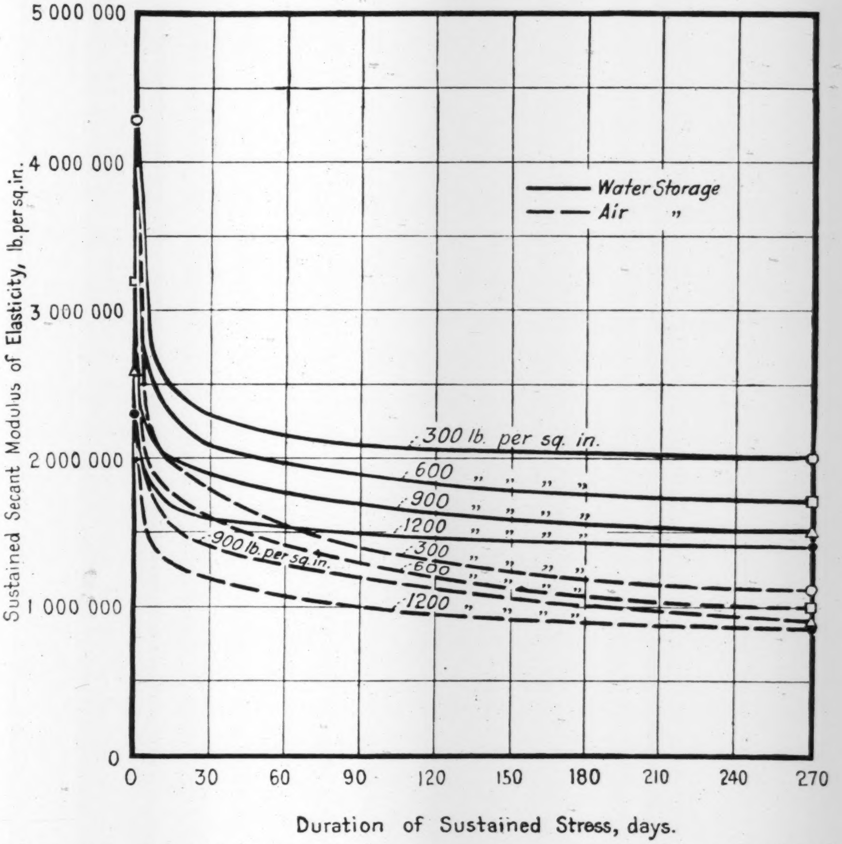

FIG. 19.—The Effect of Sustained Stress upon the Sustained Modulus of Elasticity, Stress Applied at Three Months.

In this way proper corrections to be applied to the observed deformations of the loaded specimens were determined so that the flow at any date represented the net deformation due to the sustained load prior to that date and did not include shrinkage of the concrete in the case of specimens stored in air or expansion of the concrete in the case of specimens stored under water.

The total deformation under the action of sustained load is here taken as the sum of the instantaneous deformation and the flow; that is, the volume changes due to causes other than stress are eliminated from consideration. The ratio of the sustained unit stress to the total unit deformation, is designated as the sustained secant modulus of elasticity. Just as the instantaneous secant modulus is designated as applying to a particular unit stress, for example, 800 lb. per sq. in., so the sustained secant modulus is designated as applying to a particular unit stress at the end of a definite period of sustained load, for example, 800 lb. per sq. in. after 6 months.

The results of the tests over a period of nine months of sustained load are shown graphically in Figs. 17, 18, and 19, which give the relation between the sustained modulus and time, for various unit stresses.

The effect of early loading upon the sustained modulus is shown in Fig. 17 for three groups of specimens stored under water, one group being stressed to 200 lb. per sq. in. at the age of 2 days, and the other two being loaded at the age of 7 days to the respective stresses of 300 and 600 lb. per sq. in. It will be observed that the sustained modulus decreases rapidly during the first 10 or 15 days, but after the first month the decrease is very small, though it appears that flow equilibrium had not been reached at the end of 9 months. For the specimens loaded at 7 days the sustained modulus after 270 days at the stress of 600 lb. per sq. in. is 500,000 lb. per sq. in., and at the stress of 300 lb. per sq. in. is approximately 850,000 lb. per sq. in., which values are about 0.6 those of the corresponding instantaneous moduli. The modulus of the group stressed to 200 lb. per sq. in. at 2 days decreased from the instantaneous value of 600,000 lb. per sq. in. to the sustained value of 400,000 lb. per sq. in. after 15 days.

The behavior of specimens loaded at the age of 28 days is shown in Fig. 18, the full lines giving values of the sustained modulus for cylinders stored under water and the dash lines giving values of sustained modulus for cylinders stored in air. It will be noted that again there is a rapid decrease in the modulus during the first few days of sustained load, but that at the lowest stress (300 lb. per sq. in.) the modulus at the later periods is decreasing at a much more rapid rate than it is for the highest stress (900 lb. per sq. in.). This is true for both water and air-stored groups. In other words, while the sustained secant modulus after a short period of sustained stress, say 15 days, is very much greater for a low stress than for a high one, yet after a long period of sustained stress, the difference between values at high and low stresses is much less marked.

Figure 19 shows similar diagrams for specimens loaded at the age of 3 months, the unit stresses for the several groups, both water and air storage, ranging from 300 to 1200 lb. per sq. in. It will be observed that values of the sustained modulus drop very rapidly at first, and that at the early periods of sustained stress the divergence between the moduli for high and low stresses is much less marked than was the case with specimens loaded at 28 days.

It is important to observe that the sustained modulus after a prolonged period of sustained load is but a fraction of the instantaneous modulus, whether loading be at the age of 28 days (Fig. 18) or 3 months (Fig. 19), and this fraction is in every instance greater for high stresses than for low ones. For example, from Fig. 19 the instantaneous modulus at 300 lb. per sq. in. is 4,300,000, while the corresponding sustained value after 270 days' water storage is but 2,000,000. And the corresponding moduli at 1200 lb. per sq. in. are 2,200,000 and 1,400,000.

Perhaps the most striking fact brought out by the tests is that, other things being equal, the flow of concrete under sustained stress in water storage is materially less than it is in air storage, or to put it in a different way, the sustained secant modulus, regardless of stress, is greater for a wet concrete than it is for the same concrete air dry. This is demonstrated by both Figs. 18 and 19 where the moduli curves for both air and water-stored groups are shown. Thus by consulting Fig. 19 it is seen that the sustained modulus after 270 days at a stress of 600 lb. per sq. in. is 1,700,000 for the group in water storage and 1,000,000 for the group in air storage.

SUMMARY OF RESULTS

Modulus of Elasticity:

The ratio of unit compressive stress to corresponding strain occurring immediately upon application of stress is here termed the secant modulus of elasticity or simply modulus of elasticity, without qualification.

The ratio of unit compressive stress to corresponding strain produced by the given stress constantly sustained over a considerable period of time is called the sustained modulus of elasticity.

The results of compression tests on concrete having to do with the effect upon the modulus of elasticity of age, richness of mix, repeated loads, sustained loads and moisture conditions are given below.

1. The modulus of elasticity, under the conditions of these tests, increases with age for a considerable period of time. At the end of 3 years in damp-sand storage there is no evidence that the maximum value has been reached, though the rate of increase is so small as perhaps to be negligible (see Figs. 4 and 11).

2. The increase in the modulus of elasticity is rapid at the early ages and its rate decreases quite uniformly with time. At the later ages the moduli for rich mixes continue substantially to increase after the moduli for the lean mixes have become practically constant (see Fig. 4).

3. It appears that, other things being equal, concrete for which the coarse aggregate is a dense metamorphic sandstone has a higher modulus of elasticity than does a concrete for which the aggregate is crushed granite (see Figs. 4 and 11).

4. The secant modulus of elasticity decreases as the stress increases, but for the richer mixes the variation with stress becomes less as time goes on (see Figs. 5 and 12).

5. In general the richer the mix, other things remaining constant, the higher the modulus of elasticity (see Figs. 6 and 16). But under certain conditions at least, an increase in the cement ratio produces a decrease in the modulus as is indicated by Series 1 for which the moduli for the 1:3½ mix, except at the later ages, are less than the corresponding values for the 1:4½ mix (see Fig. 6).

6. At the later ages the secant modulus continues proportionally to increase more rapidly for rich mixes than for lean ones (see Figs. 6 and 16).

7. There appears to be no direct relationship between modulus of elasticity and compressive strength which is generally applicable. But with conditions constant as regards aggregate, cement ratio and storage it appears that in a general way the higher the ultimate strength, the greater the secant modulus at a given stress (see Figs. 7 and 8).

8. Other things remaining equal, the modulus of elasticity of a given concrete when the pore spaces are filled with water is materially greater than for the same concrete when air dry (see Fig. 16). That is, while the effect of free moisture within the mass is to decrease the compressive strength, the modulus of elasticity is thereby increased.

9. The effect of a load applied at an earlier age is to increase the modulus of elasticity of the concrete when the same load is applied at a later age (see Fig. 13).

10. A sustained compressive stress, even though small, produces a flow of concrete in the direction of stress over a considerable period,

hence the sustained modulus of elasticity decreases with time (see Figs. 17, 18, and 19).

11. When the sustained load is applied at very early ages the sustained modulus soon reaches a state of practical equilibrium (see Fig. 17), but when load is first applied at later ages the sustained modulus, though taking its major decrease during the first few days continues substantially to decrease for months (see Figs. 18 and 19). For specimens loaded at 2 and 7 days the sustained values after practical equilibrium is reached are about 0.6 the corresponding values existing immediately upon application of load.

12. Between extremes of stress there is less difference in the sustained moduli after a long period of sustained stress than during the first two or three weeks following application of load (see Figs. 18 and 19).

13. The sustained modulus after a prolonged period of sustained stress is but a fraction of the instantaneous modulus, and this fraction is greater for high stresses than for low ones (see Figs. 18 and 19).

14. The sustained modulus of elasticity is substantially higher for a wet concrete than for the same concrete when air dry (see Figs. 18 and 19).

Poisson's Ratio:

Values of Poisson's ratio are determined by dividing the unit lateral expansion taking place immediately upon application of load by the corresponding unit axial compressive deformation. Following is a summary of the important findings having to do with this ratio.

15. Poisson's ratio increases with age, but at a gradually decreasing rate, so that after a year the further increase is small (see Figs. 9 and 14).

16. Poisson's ratio seems not materially to vary with either the richness of the mix or the character of aggregate (see Figs. 9 and 14).

17. Poisson's ratio is greater for low stresses than for high ones (see Figs. 10 and 15).

18. The effect of repeating at a later age a stress applied at an earlier age is to increase Poisson's ratio (see Fig. 15).

19. For the tests here considered at a stress of 800 lb. per sq. in., Poisson's ratio at one month is 0.15 to 0.17, at 6 months is about 0.19 and at one year and beyond is about 0.20, where loads are not repeated (see Figs. 9 and 14). At the age of 100 days, Poisson's ratio lies between 0.19 and 0.21 at a stress of 200 lb. per sq. in. and between 0.18 and 0.19 for stresses above 1000 lb. per sq. in. (see Fig. 10.)

DISCUSSION

MR. F. E. RICHART[1] (*presented in written form*).—The subject of this paper is one upon which much has been published, though frequently conflicting results and opinions are encountered. The determination of the modulus of elasticity and Poisson's ratio for concrete requires a high degree of precision in the measuring apparatus and in its attachment to or contact with the test specimen. The description of the methods used in this paper, including details such as humidity control, correction for temperature and shrinkage effects, types of extensometers and gages, all give assurance as to the careful methods and precision of measurement employed. While some of the results seem rather startling, the data presented are quite convincing.

Conclusion 5 of the paper indicates that in general the richer the mix, the higher the modulus of elasticity. Data of 28-day tests of 6 by 12-in. cylinders of neat cement and of concrete made by the writer some years ago indicated that the initial modulus of elasticity of neat cement ranged from about 3,500,000 lb. per sq. in. at a water-cement ratio of 0.32 down to about 1,600,000 lb. per sq. in. at a water-cement ratio of 0.75, while in ordinary concretes such as a 1:5 mixture, values of the modulus were 4,300,000 lb. per sq. in. and more. Considering that the modulus of elasticity of the aggregate materials is generally considerably higher than any of the above values it seems evident that mixtures of high aggregate content, but containing sufficient cement paste to produce a homogeneous mass, should produce concrete with a high modulus of elasticity.

It seems desirable that the term Poisson's ratio should be applied only to the ratio of elastic axial and lateral strains, or, in other words, to the ratio of the initial values of these strains for concrete. It is surprising that the values of the ratio of lateral to longitudinal strains are found to decrease as the stress increases from 200 to 1000 lb. per sq. in. Various tests have shown initial values of the ratio to be smaller than those given in the paper for medium concrete mixes, varying from 0.10 to 0.15 and gradually increasing as the load is increased. Undoubtedly much depends upon the method of measurement employed, and for any method the relative error is greatest for initial values of the ratio where stresses and strains are small. The mirror extensometer used by the authors is a precise type of instrument. In tests by Johnson,[1] who used this type of instrument, values of the ratio considerably smaller than those given by the authors were found; further, he did not report the reversal of curvature for lateral stress-strain curves at low loads such as are indicated in Fig. 3. Without disputing the accuracy of the curves of Figs. 10 and 15 it may be said that the decrease in the ratio of strains with increase of stress up to 1000 lb. per sq. in. is relatively small and probably has been given undue emphasis by the authors. Conclusion 17, that "Poisson's ratio is greater for low stresses than for high ones" is evidently based on Fig. 10, wherein the ratio decreases only up to a stress of about 800 lb. per sq. in. on concretes having ultimate strengths of 3000 to 6500 lb. per sq. in., and on Fig. 15, in which the ratio is seen to decrease up to a stress of 1000 lb. per sq. in. on 1:5 concretes having a water-cement ratio of 1:0 and presumably of fairly high strength. Here the stresses of 800 to 1000 lb. per sq. in. are evidently the "high" stresses to which the authors refer.

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[1] Research Associate Professor of Theoretical and Applied Mechanics, Engineering Experiment Station, University of Illinois, Urbana, Ill.

Tests of 1:2.1:2.5 gravel concrete having an ultimate strength of 2130 lb. per sq. in. at 28 days in which the ratio of lateral to longitudinal deformation was determined clear to the maximum load[2] showed a fairly constant value of the ratio of 0.10 to 0.13 up to about 50 per cent of the ultimate load, then a gradual increase in the ratio until at 75 to 85 per cent of the ultimate load a very rapid increase in the ratio began. At this stage it was evident that internal splitting of the concrete had started and the resulting bulging has no great significance except that it was produced without any visible surface cracking of the concrete. Shortly before failure, vertical surface cracks appeared, the ratio of lateral to longitudinal strains reached values greater than 0.5, and a combination of splitting and shearing failure ensued. For these tests, it must be said that the ratios of lateral to longitudinal strains were far greater at high stresses than at the beginning of loading.

MESSRS. HERBERT J. GILKEY[3] AND FREDRIK VOGT[4] (*by letter*).—

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[1] A. N. Johnson, "Direct Measurement of Poisson's Ratio for Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, p. 1024 (1924); also *Public Roads*, February, 1929.

[2] *Bulletin 190*, Engineering Experiment Station, University of Illinois.

[3] Associate Professor of Civil Engineering, University of Colorado, Boulder, Colo.

[4] Of Trondhjem, Norway, formerly with U. S. Bureau of Reclamation, Denver and Boulder, Colo., on investigation of arch dam models.

It is unfortunate that a paper dealing with such fundamental research material must be so brief that the data and conclusions are forced to take the form of generalized finality. Granted that the tests are well controlled and the data carefully taken (and there are few laboratories or experimenters that are better able or more likely to turn out dependable work), it is nevertheless true that different analysts will rarely place the same interpretations upon identical data.

In the paper by Davis and Troxell all of the diagrams are of the smoothed-out generalized form except Figs. 6, 7, 8 and 16. Of these again, the three plotted points for each of the curves of Fig. 6 are taken from Fig. 4. In like manner, the constellations of Figs. 7 and 8 and the two-point curves of Fig. 16 come from other plotted data (that do not appear in the paper) since the moduli of elasticity are obtained from stress-strain diagrams that are themselves graphical averages of plotted data. Thus in the entire paper, none of the original evidence appears, due no doubt to space limitations.

Referring to the several conclusions as they appear at the end of the paper, conclusions 1, 2 (first half), 4, 5, 7, and 8 (as regards strength) are in the main statements that find general acceptance and which bear out the work and conclusions of former investigators.

Conclusions 2 (second half) and 6 appear to be identical statements. They seem reasonable since the richer the mix, the more there is of the material that is undergoing change with age, namely, the cement.

While conclusion 3 is in the nature of a specific statement for the specific materials used, it seems not unreasonable to tentatively generalize it to the extent of suggesting that "in the same cement paste, low modulus aggregates may be expected to lower the modulus of elasticity of the concrete." Experiments by one of the writers have shown that this is evidently the case.

Although the statements in conclusion 5 are probably correct since they are in agreement with other published findings,[1] there seems to be some question as to the validity of drawing such conclusions from these tests. There were evidently two variables present, namely, the ratio of aggregate to cement and the ratio of water to the cement. No data relating to the water-cement ratios appear to be given for Series 1. On page 684, however, constant consistency as indicated by an approximate 1-in. slump is mentioned. For varying cement-aggregate ratios at constant consistency the water-cement ratio must have increased with the quantity of aggregate. If this were the case the modulus may have been altered either because of the increased proportion of aggregate or because of the extra water used. Probably it was due to both. To imply that the difference is due either solely or mainly to the change in quantity of aggregate seems to be unwarranted.

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[1] Stanton Walker, Portland Cement Assn. and Lewis Institute *Bulletin 5*, Table 13, is one of several possible references.

In the realm of rich mixtures in which the bond between cement paste and aggregate is not a determining factor it is reasonable to expect that the modulus of elasticity might even decrease with added cement if the modulus of the neat cement is lower than the modulus of the aggregate. Walker's[1] tests bear this out. This follows from the same reasoning as the preceding tentative generalization of conclusion 3.

Conclusion 8 is in complete agreement with results obtained by the writers. The fact that for similar concretes, that which has been air-dried prior to test is stronger than concrete that is saturated at test has been often mentioned by one of the writers and by others.[2,3] Tests at the University of Colorado not only furnish added evidence of compressive strength increase due to drying out but of increases in tensile, flexural and torsional strength. Moreover, they bear out the truth of the statement that air drying lowers the modulus of elasticity even though it increases the strength. So far as the writers are aware, Davis and Troxell are the first to have called attention to this very interesting fact. In other words, for similar concrete with like periods of moist curing, that which is air dry at test is tougher and stronger, but that which is moist is stiffer just as corn stalks or leaves are stiffer when the cells or pores are full of water. The air-dry concrete takes more deformation prior to failure. The stress-strain diagram does not rise so rapidly, but extends further and eventually rises higher than does that for saturated concrete. Forced or oven drying seems to produce a partial break-down and does not give as great an added strength as does air drying. This is the indication from a few tests by one of the writers.

The Colorado tests include specimens wet-cured and both wet and air dry at test as well as specimens re-soaked after air drying and others air-cured only and both wet and dry at test. While this is not the place to report on these tests, the rule continues to hold true that for similar specimens, those which are air dry at test will be stronger than those which are saturated. This is true in compression, flexure, tension, and torsion, but in varying degrees.

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[1] Portland Cement Association, *Bulletin 5*, Table 13.

[2] *Proceedings*, Am. Concrete Inst., pp. 424, 428 (1926); *Proceedings*, Am. Concrete Inst., pp. 168, 170 (1928); *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 425 (1927); *Transactions*, Am. Soc. Civil Engrs., Vol. 91, p. 87 (1927).

[3] W. K Hatt, *Transactions*, Am. Soc. Civil Engrs., Vol. 89, p. 271 (1926). Covers the point for both compression and flexure.

Whether the data now being worked up at the University of Colorado will bear out conclusion 9 cannot be stated. It has been noted that specimens tested to 50 or 75 per cent of their ultimate strengths, and retested again at later age after added moist curing, have moduli that closely approximate companion specimens not partially tested at earlier age. Immediate retests give lowered moduli of elasticity which agree with results by others.[1] The authors do not appear to have reported on any immediate retests.

Conclusion 10 is in agreement with previous reports by the authors and is also supported by the Colorado data. Both the amount of flow and its apparent duration are much greater if the specimen remains in ordinarily dry air than if it is immersed in water. Tests of flexural flow in air and water respectively give results similar in nature to those from compressive flow which was the type studied at California. Both kinds were employed in the Colorado tests.

Possibly it is because the Colorado air was usually drier than the 70 per cent humidity of the California laboratory, that the difference between the flow for wet and dry storage under load was even more pronounced than in the California tests.

The practical equilibrium for the sustained modulus when load is applied at early ages, referred to in conclusion 11 and shown in Fig. 17, evidently refers to specimens stored in water after loading. While this finding is verified by Colorado tests, no early equilibrium is reached if the loaded specimens remain in dry air instead of water. Nevertheless even for air storage the flow is accomplished faster in green concrete than in old concrete. The California findings pertain to compressive flow. The Colorado tests are for both compressive and flexural flow. The results agree.

Conclusion 12 is supported. Does conclusion 13 say in effect that the stress-strain diagram for a long-time test is straightened out? This appears reasonable. The Colorado tests may throw added light on this point, but the results are not yet in shape for use in this connection.

Both flexural and compressive flow test specimens kept continuously immersed and loaded to 25 and 50 per cent of their ultimate supporting power at the time of loading (as determined by short-time tests in compression and flexure on companion specimens) bear out the correctness of conclusion 14. In fact, the flow for the immersed, loaded specimens practically ceased in a relatively few days, while companion specimens loaded in air continued to flow for months.

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[1] J. W. Johnson, Fig. 21, *Bulletin 90*, Iowa State College.

Poisson's Ratio.—The range of values of Poisson's ratio agrees very well with those the writers have obtained from continuous application of load with semi-autographic lateral and longitudinal stress-strain records.

Considering the probable sizes of the ever-present experimental discrepancies, the writers question whether the order of magnitude of the differences discussed in conclusions 15, 17, 18 and 19 is not too small to be convincingly significant. Somewhat detailed consideration of this point may bring out more clearly what is meant.

On page 682, it is stated that lateral unit deformations may be measured to millionths. Assuming that the horizontal scales of Fig. 3 are unit deformations (or deformations in inches per inch, as sometimes redundantly stated, instead of "Deformation, in." as it is labelled), the following lateral unit deformations are given:

UNIT STRESS, LB. PER SQ. IN.	LATERAL UNIT DEFORMATION	PERCENTAGE ERROR DUE TO 0.000,001 ERROR IN LATERAL UNIT DEFORMATION
100	0.000 006 ±	16 ±
200	0.000 010	10
400	0.000 017	6

Although the lateral strains are the most difficult to measure accurately in Poisson's ratio determinations, they constitute but one of many sources of experimental errors of varying degrees. Considering only this source of possible error it will be seen that the entire spread in Fig. 10 at 200 lb. per sq. in. is accounted for within the range of lateral strain measurement. Moreover, more than two-thirds of the entire range from 0.18 to nearly 0.21 could be due to an error of but one-half millionth either way in unit lateral deformation. In like manner the reduced percentages of error will largely cover the spreads at higher stresses because they also are less. There is certainly question whether there can be any dependable experimental basis for attempting to define the shape (whether straight line or curved) of the lateral diagram for stresses below 200 or 400 lb. per sq. in. Even in the higher stress ranges it would seem that the tolerances for Poisson's ratio should be considerably above the narrow range that must have been assumed in attempting to lay down such definite laws regarding its variation. In addition to the foregoing, it is invariably true that the observations near zero are very unreliable. Inertia, lost motion or slack, shock and possibly conditions within the specimen itself introduce enough variable factors to make the actual direction of the stress-strain curve at the origin more a matter of "who did the drawing" than of "what the material might be." Notwithstanding the fact that probably no such tests as these have ever been conducted with greater care and competence, and giving due regard to the precautions noted on page 684, the writers are convinced that the unsatisfactory "zero zone" was only narrowed. It was not removed as evidenced by the preceding computation based on the possible error due to a single element as given by the authors. This assumed not more than one-half millionth either way. If plus or minus a millionth was meant, the spread would be doubled.

While the writers do not profess to have attained as high precision in their tests, they have found nothing in their results that tends to bear out conclusion 17. The indication is, if anything, the other way. Certainly Poisson's ratio or its non-elastic equivalent increases very rapidly in the higher stress territory. Figure 10, in its right-hand portion shows something of this but to nowhere near the same extent as is shown in the Colorado tests. The lateral swelling due to wedging and splitting action becomes relatively much more important and pronounced than is the longitudinal deformation in the later stages of test.

In the mixture used in these tests the concrete was 3200 lb. per sq. in. at 28 days standard curing. Approximate average values for Poisson's ratio for 28 days and older were: 0.15 to 18 at low stresses say below 1000 lb. per sq. in. and 0.21 at 2000 lb. per sq. in. The ratio attains 0.40, 0.50 or even higher values for stresses near the ultimate. For 7-day specimens, Poisson's ratio may start to increase rapidly from 500 lb. per sq. in. and upwards. These results are typical of other mixtures. They are not in agreement with the statements in the third paragraph on page 689.

While the California tests were devoted primarily to compression, the Colorado tests, as has been previously stated, included flexural, and a little tensile flow as well as compression. The general findings are in agreement.

The flexural tests on beams in air after six months gave flows proportional to the stress, regardless of the maximum stresses in the beams, which varied from 75 to 373 lb. per sq. in. and regardless of age at loading, which varied from 7 days to 3 months. Unit flow was approximately 1.6 millionths for each lb. per sq. in. stress. Beams loaded in air deflected as much as five times the initial deflection before reaching relative quietude after a period of three months or more.

The writers commend the authors upon the splendid work they are doing and hope to see it continued. The purpose of this discussion is not to detract from the paper in any way but rather to offer corroborative evidence when it could be given and to raise questions regarding possible uncertainties. Space does not permit more than rather dogmatic unsupported statements, which is one of the major criticisms of the paper itself. The unsupported evidence offered is not intended as final. It is introduced in an attempt to forestall premature general acceptances of some items that still involve elements of doubt. When so much new territory is being explored, it is not to be expected or even hoped that some controversy, checking over, and even back tracking will not be in order before general agreement is attained and final truth established.

MESSRS. R. E. DAVIS[1] AND G. E. TROXELL[2] (*authors' closure by letter*).—The discussions of the paper have served to corroborate some of the findings and have emphasized a number of interesting facts. The authors are grateful to Messrs. Gilkey, Vogt and Richart for their contributions.

Mr. Richart calls attention to the fact that the stiffness of the cement paste as well as that of the aggregate has an important bearing upon the modulus of elasticity of concrete. This is illustrated by the curves of Fig. 6, which show that at the earlier ages the modulus of elasticity for the 1:3.5 mix is considerably lower than for the 1:4.5 mix, but that with the passage of time, as the cement paste hardens, the modulus of elasticity of the rich mix gradually approaches that of the 1:4.5 mix, finally becoming the greater.

Mr. Gilkey states with regard to the variation of modulus of elasticity with richness of mix, that it is impossible to say whether this variation is due to change in quantity of cement or to change in quantity of mixing water. Inasmuch as for the tests under discussion the volume of mixing water was not kept constant, but was varied so as to produce a constant consistency, this is in a measure true; but since the actual water content of a given volume of concrete, whether lean or rich, varied within comparatively narrow limits, it would appear that the variation in the modulus of elasticity between the several mixes was primarily due to variations in cement content rather than in water content.

Conclusion 17, that "Poisson's ratio is greater for low stresses than for high ones" ought, perhaps, to be modified. For the concretes employed in these investigations—which included some in which the aggregates were all of granitic character, and others in which the coarse aggregate was a metamorphic sandstone and the fine aggregate a quartz sand—Poisson's ratio was distinctly greater at stresses below 200 or 300 lb. per sq. in. than at stresses of intermediate value, say, 800 to 1000 lb. per sq. in. With further increase of stress, Poisson's ratio increased but slightly up to about one-half the ultimate strength of the concrete, and then much more rapidly, becoming large as the ultimate strength was approached. From a very considerable number of curves for individual specimens, it appears that this variation of Poisson's ratio with stress within what may be considered the normal range of working stresses, becomes considerably less with age. In other words, the curve showing the relation between Poisson's ratio and stress is more nearly a straight line at later ages than at early ones.

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This decrease of Poisson's ratio with increase in stress for the lower stresses seems to be a property of the concretes with which we were dealing, since for other tests made later, for which the aggregates were Coast Range gravels and the cement was Atlas Lumnite, Poisson's ratio was practically constant for stresses of less than 100 lb. per sq. in. up to about one-half the ultimate strength.

Concerning Mr. Gilkey's comments as to the possibility of observational errors at low stresses being so large as to render insignificant the differences discussed in conclusions 15, 17, 18 and 19, an inspection of the individual lateral and axial stress-strain curves indicates that the observational errors were so small as to be inconsequential for stresses above 200 lb. per sq. in. It is estimated that at this stress the probable error of observation did not exceed 3 per cent.

Conclusion 13 might be amplified. Under the action of continuous compressive stress the sustained modulus, after a prolonged period of sustained load, is much smaller than the corresponding modulus, and the lower the magnitude of the sustained compressive stress, the larger relative decrease in the sustained modulus.

THE WATER ABSORPTION AND PENETRABILITY OF BRICK[1]

By J. W. McBurney[2]

Synopsis

This paper records the values obtained from the determination of water absorption and penetrability by a number of methods on a variety of makes of brick representing a considerable range in properties and methods of manufacture. The value of water absorption as a measure of resistance to frost action is discussed. The mechanism of water transmission through masonry is discussed from the standpoint of absorption properties of the units.

On the basis of the data recorded and field observations it is considered that the water absorption or apparent porosity of brick is not a measure of resistance to climatic action in so far as differences in raw materials and methods of manufacture are involved. However, for the same clay or shale and for the same method of forming, the difference between satisfactory and unsatisfactory weathering performance may usually be distinguished by difference in water absorption.

It is believed that while Schurecht's ratio (water absorption, 48-hour cold immersion to water absorption, 5-hour boiling) is a much better measure of disintegration through freezing and thawing than is water absorption by any one method, it is not sufficiently reliable to furnish a basis for classification.

Introduction

The present paper is an attempt to do four things:

1. To record the values obtained from the determination of water absorption and penetrability by a number of methods on a variety of makes of brick representing a considerable range in properties and methods of manufacture.

2. To determine whether such regular and constant relations exist among any of the various measures of water absorption and penetrability that it would be possible to convert the results of one method of test to the results of another method.

3. To determine, if possible in the present state of our knowledge, whether water absorption affords any guide to the rating of bricks from the standpoint of resistance to disintegration through frost action.

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[1] Publication approved by the Director of the Bureau of Standards of the U. S. Department of Commerce, Washington, D. C.

[2] Research Associate of the Common Brick Manufacturers' Association of America at the U. S. Bureau of Standards, Washington, D. C.

4. To discuss, or rather to speculate on, the possible bearing of water absorption and penetrability of bricks on the problem of water transmission through masonry.

SOURCE OF SAMPLES

The data here presented are the results of determinations of the absorption of brick specimens from the investigation reported in a paper entitled "The Compressive and Transverse Strength of Brick"[1] presented before the American Ceramic Society. The sample numbers in both that and the present paper refer to the same bricks. For a detailed description of these samples reference is made to the preceding paper.

For the purposes of the present paper it is sufficient to note that 39 samples representing 27 brick plants were tested. The numbers 1 to 27 refer to the plants. Where samples with the same number are distinguished by letters *A* and *B* or *A*, *B* and *C*, these letters refer to samples classified by the manufacturers as "well burned" and "salmon" or "clinker," "body" and "salmon" respectively (see previous paper[1]). This question of manufacturer's classifications will be discussed later.

Nos. 1 and 3, 17 to 22 inclusive and 24 are reported to represent the product of shale brick plants. All other numbers represent clay specimens.

METHODS OF TESTS

All bricks as received were dried in a gas-fired oven for 96 hours. The temperature of the oven varied between 110 and 140° C. The time for subsequent redrying was that necessary for attaining the original dry weight. In general 48 hours sufficed. The procedure finally adopted was as follows: The samples were removed from the oven and left in the laboratory over night. In the morning they were weighed on a balance sensitive to ± 2.5 g., then set on end in $\frac{1}{8}$ in. of water at room temperature for one hour. If, during the hour, the water rose the entire length of the brick and wetted the upper surface the time was noted. Usually the exact moment of appearance of water on the top could be determined by the change in the appearance of the brick when wetted. Some dark colored bricks required the determination of this point by touch. The use of mixtures of powdered dye and sugar as developed by Carson[2] for detecting water transmission through paper was tried but apparently added nothing to the sensitivity of the test.

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[1] J. W. McBurney, "The Compressive and Transverse Strength of Brick," *Journal*, Am. Ceramic Soc., Vol. 12, p. 217 (1929); also U. S. Bureau of Standards *Journal of Research*, Vol. 2, No. 4, p. 821 (1929).

[2] F. T. Carson, "Testing Paper for Permeability to Liquids," *Paper Trade Journal*, March 5, 1925.

At the end of the hour the brick was removed from contact with the water, the surface water wiped off with a damp cloth and the brick weighed. The percentage of absorption was calculated from the formula $100\frac{(b-a)}{a}$, where a equals the weight of the dry brick and b the weight of the brick plus absorbed water. The brick was then redried in the oven, cooled and the test repeated except that the face instead of the end of the brick was in contact with the water. After again redrying, the rest was repeated except that the flat of the brick was the contact face.

The description, so far, applies to the procedure used in the determination of what is hereinafter referred to as penetrability. Water absorption determined by partial immersion is considered a measure of penetrability.

After a final redrying, the total immersion series was started. Here the sample was totally immersed in water at room temperature for a period of one hour, then taken out and the surface water wiped off with a damp cloth, weighed, and put back in the water for a further period of four hours. After again weighing, 19 additional hours' immersion were given. Then an additional 24 hours were provided. This gave weighings at 1, 5, 24 and 48 hours of total immersion. Without being dried the bricks were immersed and the water brought up to boiling within one hour and boiling was continued for 5 hours. The bricks and water were permitted to cool to room temperature over night and the next morning the bricks were wiped and weighed as before. The percentage of absorption was recorded as that for a 5-hour boil.

As previously mentioned, this procedure was the one finally adopted. A number of the earlier tests did not include the 1-hour total immersion period. Instead, the 1-hour partial immersion was made the start of the 5, 24 and 48-hour total immersion test. The possible effect of this change of procedure on the water absorption values for the 5, 24 and 48-hour periods will be discussed later.

A few samples were tested according to the following procedure: A circle, approximately 2 cm. in diameter, was drawn on the face of the brick with a wax pencil. One milliliter of water was added from a pipette, time of delivery approximately two seconds, and the length of time for the water to be completely absorbed was observed.

Presentation of Data

Table I gives the water absorption by weight expressed as a percentage of the dry weight of the brick at periods of 1, 5, 24 and 48 hours' total immersion in cold water as well as the percentage of water absorbed during 48 hours' cold immersion plus 5 hours' boiling. It will be noted that for a number of the samples no determination was made of the water absorption at the end of 1 hour total immersion. For purpose of comparison, Table I includes the percentage absorption of all samples after one hour partial immersion flat.

Table I.—Water Absorption of Bricks.

(Percentage of Dry Weight.)

Sample	Method of Manufacture	Raw Material	Number of Tests	Partial Immersion Flat	Total Immersion					Ratios		
					1-Hour Cold	5-Hour Cold	24-Hour Cold	48-Hour Cold	5-Hour Boil	5-Hour Cold to 48-Hour Cold	5-Hour Cold to 5-Hour Boil	48-Hour Cold to 5-Hour Boil (Schurecht's Ratio)
No. 1	Dry press	Shale	3	6.9[a]	....	7.1	7.3	7.4	9.9	0.96	0.72	0.75
No. 2A		Clay	50	15.9[a]	....	16.0	16.1	16.3	22.0	0.98	0.73	0.74
No. 2B		Clay	6	17.9[a]	....	18.2	18.6	19.1	20.7	0.96	0.88	0.92
No. 3		Shale	5	3.29[a]	....	3.3	3.4	3.4	5.6	0.96	0.60	0.62
No. 4A	Semi-dry press	Clay	3	5.8[a]	....	6.1	7.2	7.5	9.9	0.81	0.62	0.76
No. 4B		Clay	3	7.2[a]	....	7.5	8.5	8.8	10.9	0.85	0.69	0.81
No. 4C		Clay	3	14.1[a]	....	14.7	15.3	15.5	16.8	0.95	0.88	0.92
No. 5	Soft mud.	Clay	5	8.8[a]	....	9.5	10.2	10.4	16.8	0.91	0.56	0.62
No. 6		Clay	50	17.4[a]	....	20.0	20.5	20.7	22.9	0.97	0.87	0.90
No. 7		Clay	50	13.8[a]	....	14.4	14.6	14.8	19.2	0.97	0.75	0.77
No. 8		Clay	3	23.1[a]	19.9	20.5	21.1	21.8	27.6	0.94	0.74	0.79
No. 9		Clay	3	21.0[a]	20.5	20.9	21.8	22.3	25.3	0.94	0.83	0.88
No. 10		Clay	1	11.0[a]	10.1	10.3	10.9	11.1	15.1	0.93	0.68	0.74
No. 11		Clay	1	10.0[a]	10.0	10.0	10.4	10.9	16.9	0.92	0.59	0.65
No. 12		Clay	1	17.6[a]	17.4	17.6	18.1	18.3	23.7	0.96	0.74	0.77
No. 13A		Clay	75	5.0	....	6.3	6.6	6.9	9.6	0.91	0.66	0.72
No. 13B		Clay	6	14.0	....	18.6	18.8	18.9	19.2	0.98	0.97	0.99
No. 14A	Stiff mud, end cut	Clay	25	5.9[b]	....	10.4	12.0	12.6	17.5	0.83	0.59	0.72
No. 14B		Clay	5	20.4[a]	21.9	23.1	23.7	23.8	26.2	0.97	0.88	0.91
No. 16A		Clay	5	7.4	7.8	8.0	9.0	9.4	13.2	0.85	0.61	0.71
No. 16B		Clay	5	10.1[a]	9.7	10.1	10.8	11.4	14.7	0.89	0.69	0.78
No. 16C		Clay	4	16.2[a]	15.8	16.4	18.0	18.6	21.4	0.88	0.77	0.87
No. 17A	Stiff mud, side cut	Shale	50	2.8	....	5.3	5.9	6.2	8.7	0.86	0.61	0.71
No. 17B		Shale	6	9.8[a]	....	10.2	10.6	11.0	12.9	0.93	0.79	0.85
No. 18		Shale	6	2.3	....	4.9	5.3	5.5	7.6	0.89	0.65	0.73
No. 19A		Shale	6	0.6	....	2.7	4.3	4.4	5.3	0.61	0.51	0.83
No. 19B		Shale	6	1.6	3.2	5.3	5.9	6.1	7.8	0.87	0.68	0.78
No. 20		Shale	3	5.5	....	6.7	7.2	7.6	10.0	0.88	0.67	0.76
No. 21		Shale	25	0.8	1.1	1.8	2.3	2.6	5.0	0.69	0.35	0.51
No. 22A		Shale	5	5.1[a]	4.9	5.4	6.0	6.4	8.4	0.84	0.65	0.77
No. 22B		Shale	5	8.5[a]	8.6	9.1	9.9	10.0	12.5	0.91	0.73	0.80
No. 23A		Clay	2	3.2	....	3.1	3.4	3.6	7.0	0.86	0.44	0.51
No. 23B		Clay	3	7.1[a]	....	7.6	8.5	8.8	11.5	0.86	0.66	0.77
No. 23C		Clay	3	9.0	....	15.1	15.9	16.2	18.2	0.94	0.83	0.89
No. 24A		Shale	3	1.4	2.2	2.5	2.8	3.0	4.9	0.83	0.51	0.61
No. 24B		Shale	4	10.1[a]	9.7	10.1	10.4	10.6	13.2	0.95	0.76	0.80
No. 25		Clay	1	10.0[a]	11.5	11.5	12.1	12.7	14.5	0.91	0.79	0.88
No. 26		Clay	1	7.0[a]	7.0	8.0	8.2	8.9	10.9	0.90	0.73	0.82
No. 27		Clay	1	10.7[a]	10.3	10.9	11.7	12.1	14.5	0.90	0.75	0.83

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[a] Brick wetted through during period of test.

[b] Four of twenty-five tests more than 1 hour; other twenty-one tests did not give true wetting; water appeared on opposite face through cracks and by creeping up exterior.

Table II gives the water absorption by weight expressed as a percentage of the dry weight of the brick for partial immersion on

TABLE II.—WATER ABSORPTION BY PARTIAL IMMERSION AND TIME REQUIRED TO COMPLETELY WET BRICK.

(Depth of water for all samples $\frac{1}{8}$ in. Time of exposure 1 hour.)

Sample	Method of Manufacture	Raw Material	Number of Tests	Water Absorption by Partial Immersion (Percentage of dry weight)			Time for Completing Wetting of Brick						Absorption in 1 Hour per sq. in. of Cross-section as Percentage of Dry Weight		
							End		Edge		Flat				
				End	Edge	Flat	Min.	Sec.	Min.	Sec.	Min.	Sec.	End	Edge	Flat
No. 1	Dry press	Shale	3	6.2	7.2	6.9	*a*		14	00	*b*		0.73	0.40	0.23
No. 2A		Clay	25	16.2	16.1	16.2	28	00	4	20	3	00	1.92	0.90	0.54
No. 2B		Clay	6	15.8	18.3	17.9	*c*		7	50	4	00	1.87	1.02	0.60
No. 4A	Semi-dry press	Clay	3	3.1	5.6	5.8	Ov. 60	00	Ov. 60	00	14	00	0.37	0.31	0.19
No. 4B		Clay	3	3.7	6.7	7.2	Ov. 60	00	*d*		16	20	0.44	0.37	0.24
No. 4C		Clay	3	6.1	12.8	14.1	Ov. 60	00	46	40	25	40	0.72	0.71	0.47
No. 6	Soft mud	Clay	25	5.4	11.9	17.1	Ov. 60	00	Ov. 60	00	48	00	0.64	0.66	0.57
No. 7		Clay	25	4.8	11.2	14.0	Ov. 60	00	*e*		31	00	0.56	0.62	0.47
No. 8		Clay	3	12.2	23.0	23.1	Ov. 60	00	30	00	15	00	1.44	1.28	0.77
No. 9		Clay	3	7.5	16.7	21.0	Ov. 60	00	Ov. 60	00	32	00	0.89	0.93	0.70
No. 10		Clay	1	5.3	11.0	11.1	Ov. 60	00	40	00	18	00	0.63	0.61	0.39
No. 11		Clay	1	7.8	10.0	10.0	Ov. 60	00	17	00	5	00	0.92	0.56	0.33
No. 12		Clay	1	10.7	17.6	17.6	Ov. 60	00	25	00	10	00	1.27	0.98	0.59
No. 13A		Clay	14	2.6	5.5	6.9	Ov. 60	00	Ov. 60	00	*f*		0.30	0.30	0.23
No. 13B		Clay	6	4.0	10.0	14.0	Ov. 60	00	Ov. 60	00	Ov. 60	00	0.47	0.56	0.47
No. 14A	Stiff mud, end cut	Clay	25	3.3	4.8	5.9	Ov. 60	00	Ov. 60	00	*g*		0.39	0.27	0.20
No. 14B		Clay	5	8.0	14.6	20.4	Ov. 60	00	Ov. 60	00	35	00	0.95	0.81	0.68
No. 16A		Clay	5	4.6	6.8	7.4	Ov. 60	00	*h*		*i*		0.54	0.38	0.25
No. 16B		Clay	5	5.2	10.0	10.1	Ov. 60	00	46	00	15	00	0.62	0.55	0.34
No. 16C		Clay	4	10.1	15.9	16.2	Ov. 60	00	39	00	14	00	1.20	0.88	0.54
No. 17A	Stiff mud, side cut	Shale	25	1.0	2.6	3.0	Ov. 60	00	Ov. 60	00	Ov. 60	00	0.12	0.14	0.10
No. 17B		Shale	6	2.8	7.0	9.8	Ov. 60	00	Ov. 60	00	25	00	0.33	0.39	0.33
No. 18		Shale	6	0.7	1.7	2.3	Ov. 60	00	Ov. 60	00	Ov. 60	00	0.08	0.09	0.08
No. 19A		Shale	5	0.3	0.3	0.6	Ov. 60	00	Ov. 60	00	Ov. 60	00	0.04	0.01	0.02
No. 19B		Shale	5	0.03	1.2	1.6	Ov. 60	00	Ov. 60	00	Ov. 60	00	0.03	0.07	0.05
No. 21		Shale	25	0.2	0.5	0.8	Ov. 60	00	Ov. 60	00	Ov. 60	00	0.03	0.02	0.03
No. 22A		Shale	5	2.2	4.3	5.1	Ov. 60	00	Ov. 60	00	12	00	0.26	0.24	0.17
No. 22B		Shale	5	3.4	6.5	8.7	Ov. 60	00	*j*		20	00	0.40	0.36	0.29
No. 23A		Clay	2	0.7	1.2	3.2	Ov. 60	00	Ov. 60	00	Ov. 60	00	0.08	0.07	0.11
No. 23B		Clay	3	2.6	5.7	7.1	Ov. 60	00	Ov. 60	00	8	00	0.31	0.32	0.24
No. 23C		Clay	3	1.8	4.5	9.0	Ov. 60	00	Ov. 60	00	*k*		0.21	0.25	0.30
No. 24A		Shale	3	0.2	1.2	1.4	Ov. 60	00	Ov. 60	00	Ov. 60	00	0.02	0.07	0.05
No. 24B		Shale	4	3.3	8.6	10.1	Ov. 60	00	*l*		26	00	0.39	0.48	0.34
No. 25		Clay	1	2.3	4.7	10.0	Ov. 60	00	Ov. 60	00	50	00	0.27	0.26	0.33
No. 26		Clay	1	1.9	6.4	7.0	Ov. 60	00	55	00	10	00	0.22	0.36	0.23
No. 27		Clay	1	5.4	9.7	10.7	Ov. 60	00	58	00	21	00	0.64	0.54	0.36

[a] But one of three struck through, time 10 minutes.
[b] Time not recorded.
[c] Three bricks wet within hour, average of these three 53 minutes.
[d] Two of three samples wet, average 41 minutes.
[e] Sixteen of these samples more than 1 hour.
[f] Two of fourteen specimens more than 1 hour, average of twelve is 35 minutes.
[g] Four of twenty-five tests more than 1 hour; other twenty-one tests did not give true wetting; water appeared on opposite face through cracks and by creeping up exterior.
[h] Two of five samples more than 1 hour, average of three 46 minutes.
[i] One of five samples more than 1 hour, average of four 21 minutes.
[j] One of five tests through in 50 minutes, other four more than 1 hour.
[k] Two of three tests more than 1 hour, third test through in 3 minutes (?).
[l] Two of four tests through in average of 53 minutes, other two greater than 1 hour.

end, face and flat of the samples at the end of 1 hour. If, during the hour, the brick was completely wetted, the time is recorded. The last three columns of Table II give the percentage absorption for partial immersion at the end of 1 hour divided by the area of cross-section of the brick expressed in square inches. In other words, the last three columns represent the absorption in 1 hour per square inch of cross-section expressed as a percentage of the dry weight of the brick.

TABLE III.—VOLUME AND WEIGHT OF BRICKS.

Sample	Dimensions			Gross Volume, cu. cm.	Correction of Frog, cu. cm.	Actual Volume, cu. cm.	Dry Weight, g.	Weight, g. per cu. cm.
	Length, cm.	Breadth, cm.	Depth, cm.					
No. 1........	20.1	9.15	5.8	1070	..	1070	2280	2.13
No. 2A.......	20.8	9.65	5.95	1190	..	1190	1870	1.57
No. 2B.......	20.3	9.4	5.7	1090	..	1090	1920	1.76
No. 3........	21.8	11.4	6.4	1585	..	1585	3405	2.15
No. 4A.......	21.2	10.0	5.5	1170	..	1170	2425	2.07
No. 4B.......	21.4	10.0	5.4	1155	..	1155	2455	2.12
No. 4C.......	22.0	10.4	5.6	1280	..	1280	2470	1.93
No. 5........	20.1	9.6	5.6	1080	..	1080	2075	1.92
No. 6........	20.7	9.15	6.25	1185	70	1115	1885	1.69
No. 7........	20.0	8.4	5.8	975	40	935	1760	1.88
No. 8........	20.35	9.6	5.65	1100	50	1050	1740	1.66
No. 9........	20.7	9.85	5.9	1205	60	1145	1825	1.59
No. 10.......	20.6	9.8	6.3	1270	..	1270	2375	1.87
No. 11.......	21.0	9.9	5.8	1205	..	1205	2300	1.91
No. 12.......	20.6	9.8	6.7	1355	..	1355	2250	1.66
No. 13A......	20.0	9.0	5.7	1030	25	1005	2060	2.05
No. 13B......	21.0	9.4	5.8	1145	25	1120	1920	1.72
No. 14A......	19.8	9.15	5.6	1015	..	1015	1750	1.72
No. 14B......	20.6	9.5	5.5	1075	..	1075	1780	1.74
No. 16A......	19.8	9.2	6.0	1095	..	1095	2125	1.94
No. 16B......	19.7	9.7	5.9	1125	..	1125	2170	1.93
No. 16C......	20.3	10.0	6.1	1240	..	1240	2140	1.73
No. 17A......	20.8	9.30	5.6	1085	..	1085	2385	2.20
No. 17B......	21.5	9.85	5.8	1225	..	1225	2480	2.02
No. 18.......	20.6	9.45	5.8	1130	..	1130	2525	2.24
No. 19A......	20.1	9.0	5.5	1000	..	1000	2320	2.32
No. 19B......	20.4	9.4	5.7	1090	..	1090	2395	2.19
No. 20.......	20.4	9.45	5.8	1120	..	1120	2400	2.15
No. 21.......	20.0	9.1	5.6	1020	..	1020	2410	2.36
No. 22A......	20.9	9.65	5.6	1130	..	1130	2455	2.17
No. 22B......	21.1	9.9	5.8	1210	..	1210	2405	1.99
No. 23A......	20.6	9.0	5.8	1075	..	1075	2200	2.05
No 23B......	20.7	9.15	5.7	1080	..	1080	2170	2.01
No. 23C......	21.0	9.4	5.8	1145	..	1145	2115	1.84
No. 24A......	20.4	8.95	5.6	1020	..	1020	2440	2.39
No. 24B......	21.3	9.5	6.0	1215	..	1215	2420	1.99
No. 25.......	20.8	9.9	6.1	1260	..	1260	2440	1.93
No. 26.......	20.7	9.7	6.25	1250	..	1250	2585	2.06
No. 27.......	20.7	9.7	6.5	1305	..	1305	2480	1.90

For those who prefer to consider the samples from the standpoint of apparent porosity, Table III gives the volume data with the aid of which the data of Tables I and II can be recalculated in terms of apparent porosity.

DISCUSSION OF DATA

Total Immersion Cold:

It is conceivable that the absorbing of water by bricks, totally immersed, takes place in two stages. Water travels from all sides

toward the center of the brick. Assuming equal rates of travel, the dry interior of the brick would present a form somewhat similar to an ellipsoid, gradually shrinking to a point. The reader is referred to Mahr's[1] paper for a method of observing the distribution of a liquid in a ceramic body. The disappearance of the dry interior of the brick completes the first stage of the brick's water absorption. The brick is now wetted throughout. Further absorption proceeds at a different rate, usually much slower, and can be conceived as being a searching out and filling of some of the pore spaces which were not filled during the first stage.

Table I deals largely with values for absorption during the second stage, that is, after the samples have become at least partially wetted throughout. It would seem that samples Nos. 19*A* and 21 are the only ones that possibly had not completed the first stage at the end of the 5-hour period.

Comparing first the data on the absorption in 1 hour of bricks partially and completely immersed it will be noticed that the percentage absorptions by the two methods are in general in rather close agreement. In the case of samples Nos. 8, 9, 10, 12, 16*B*, 16*C*, 22*A*, 24*B* and 27, 1 hour partial immersion flat gives a greater absorption than total immersion for the same period. This is explainable by the greater tendency to trap air in the brick when the test is conducted by totally immersing. All of these bricks, as well as sample No. 11, became completely wetted in less than one hour during the partial-immersion test. On the other hand, for those samples, Nos. 16*A*, 19*B*, 21 and 24*A*, for which the absorption by partial immersion was less than the absorption by total immersion, there was not a complete wetting by partial immersion in one hour.

Owing to the two methods (partial and total immersion) used in determining the 1-hour absorption, it is deemed proper to separate the consideration of the results, where 1-hour absorption is compared with absorptions made during longer periods of time. Five-hour total immersions gave absorptions but little if any higher than those obtained by 1-hour partial immersion when within the hour the brick had completely wetted. The dry-press sample No. 3 was the most noticeable exception to the tendency for bricks with low absorption at 1 hour to add relatively more for the 5-hour period than did those bricks for which the absorption for 1 hour was high.

The ratios, water absorption during total immersion for 1 hour to water absorption during total immersion for 5 hours, ranged from 0.604

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[1] H. W. Mahr, "Studies in the Absorption of Water by Building Brick," *Journal of Industrial and Engineering Chemistry*, Vol. 6, p. 800 (1914).

(sample 19*B*) to 1.00 (samples Nos. 11 and 25). The smallest gain in percentage absorption between 1 and 5 hours is 0.0 per cent for samples Nos. 11 and 25 and the largest 2.1 per cent for sample No. 19*B*.

The 24-hour water absorption can be dismissed by the observation that obviously it is intermediate in magnitude with reference to the 5 and 48-hour periods. Usually it is closer to the 48-hour value than to the 5-hour value.

Considering absorption determined by 48-hour cold immersion, the first question suggested is whether this is near the limiting value for cold absorption at "infinite time." On the basis of data on hollow tile given by Foster[1] who continued total immersion to 216 hours, the absorption at the end of 48 hours ranged from 87 to 96 per cent of the absorption recorded at the end of 216 hours. Douty and Gibson[2] give percentage absorption by total immersion at 17 days. The percentage of absorption at 48 hours ranges from 82 to 95 per cent of that found at the end of 17 days. Douty and Beebe[3] also give values for absorption by cold immersion up to 110 days. Comparing the water absorption at 17 days with that at 110 days, it was found that the 17-day period gave absorptions ranging from 72 to 95 per cent of that found at the end of 110 days.

From the above it is evident that the cold absorption at 48 hours does not at all represent the maximum cold absorption possible with longer time. Washburn[4] considers that owing to diffusion of air, cold, immersion will eventually fill all open pores including "blind alley" pores. However, this must be considered as an exceedingly slow process since the water is presumably saturated with air and the only forces effective for causing diffusion are those tending to compress the trapped air such as capillary and gravitational forces.

The ratio for absorption by 5-hour cold total immersion to that by 48-hour cold total immersion ranged from 0.61 to 0.98 according to the data here reported. Ratios above 0.95 characterized dry press bricks on the basis of these data. Soft mud brick ranged from 0.91 to 0.97 and the stiff mud bricks, with the exception of the "salmons" which are above 0.91, ranged from 0.91 down to 0.61.

It is very probable that the 5-hour cold to 48-hour cold total immersion ratio would be considerably influenced by the method of procedure during the first hour. Attention is called to sample No. 8 for which the absorption was greater by 1-hour partial immersion than by 48-hour total immersion.

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[1] H. D. Foster, "Effectiveness of Different Methods of Making Absorption Determinations as Applied to Hollow Building Tile," *Journal*, Am. Ceramic Soc., Vol. 5, p. 788 (1922).

[2] D. E. Douty and H. C. Gibson, "The Influence of the Absorptive Capacity of Brick upon the Adhesion of Mortars," *Proceedings*, Am. Soc. Testing Mats., Vol. VIII, p. 518 (1908).

[3] D. E. Douty and L. L. Beebe, "Some Further Experiments upon the Absorption, Porosity and Specific Gravity of Building Brick," *Proceedings*, Am. Soc. Testing Mats., Vol. XI, p. 767 (1911).

[4] E. W. Washburn, "Porosity and the Mechanism of Absorption," *Journal*, Am. Ceramic Soc., Vol. 4, p. 920 (1921).

Water Absorption by Boiling:

Whether water absorption as determined by boiling for 5 hours represents complete filling of the open pores is apparently questionable. Washburn[1] shows that it does not for a certain type of stoneware body and considers that its accuracy for certain other materials depends upon a balancing of errors.

"Federal Specification Board Specification No. 504, United States Government Master Specification for Brick, Clay, Common" provides for the determination of water absorption either by 5-hour boiling or by 5-hour cold immersion with the proviso that 5-hour cold immersion shall be considered as giving 75 per cent of the absorption resulting from 5-hour boiling. The average of the ratio of absorptions by 5-hour cold immersion to those by 5-hour boiling for all samples reported in Table I is 0.695. Eliminating the values for the samples classified by the manufacturer as salmons, the ratio average is 0.655.[2] The maximum ratio is 0.97 for sample No. 13*B*, a salmon and classed as unmarketable brick by the manufacturer and the minimum ratio is 0.35 for sample No. 21, which is probably the hardest burned brick in the list. It is evident that the ratio of absorption by 5-hour cold immersion to that by 5-hour boiling is affected both by the difference in nature of cold absorption and absorption by boiling (a function of the total number, shapes and sizes of channel pores and dead-end pores) and by the fact that with some hard-burned bricks the rate of penetrability is such that 5 hours' cold immersion is probably insufficient to give complete wetting. In other words the ratio of absorption by 5-hour cold immersion to absorption by 5-hour boil is not constant for different kinds of brick. On the basis of these data it would seem that greater accuracy could be secured by a grading of ratios corresponding to the grades of brick specified. Tentatively the following relations are suggested in place of the one ratio:

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[1] E. W. Washburn, "Porosity: III. Water as an Absorption Liquid," *Journal*, Am. Ceramic Soc., Vol. 4, p. 981 (1921).

[2] It is recognized that the terms "well burned" and "under-burned" as applied to brick have different meanings for the ceramist and for the user. These terms are used in the present paper as here defined:

"*Well burned*" or "*normally burned*" *bricks.*—Bricks so manufactured that they possess the quality of satisfactory resistance to the disintegrating effects of exposure to weather in the judgment of their maker.

"*Under-burned*" or "*salmon*" *bricks.*—Bricks so manufactured that they do not possess sufficient resistance to the disintegrating effect of exposure to weather in the judgment of their maker, so that he either considers them unmarketable, or sells them with the caution that they be used only for filler or back-up purposes.

Grade	Absorption by Boiling, per cent by weight	Ratio of 5-Hour Cold Immersion to 5-Hour Boil
Soft	Above 20	0.85
Medium	12 – 20	0.75
Hard	5 – 12	0.65
Vitrified	0 – 5	0.55

The interpretation of the ratio of absorption by 48-hour cold immersion to absorption by 5-hour boiling, will be discussed later. It is sufficient for the present to point out that this ratio ranges from 0.51 to 0.99 and that the higher values are associated with the specimens classified as "salmons" by the manufacturer while the lower values characterize the well-burned[1] samples. Mr. H. G. Schurecht[2] was the first investigator who, to the author's knowledge, made use of this ratio, hence the name "Schurecht's ratio" is proposed for this relation.

Penetrability:

Penetrability, quoting Washburn[3] "may be qualitatively defined as the ease with which a liquid is drawn into the pores of the body by capillary action, without attendant chemical action between the body and the liquid." Obviously there are a number of possible ways of estimating the relative penetrability of brick. Washburn suggests as one measure "the amount of a standard liquid which, in a stated time, a standard test piece will absorb, at a stated temperature, when a stated surface of the piece in a determined position (horizontal or vertical, and, if horizontal then a lower not an upper surface), is kept in contact with the liquid, and the air surrounding the test piece is kept saturated." Another possible measure would consist in determining the time required to completely wet through a brick partially immersed according to some standard procedure. The converse of this last method would be to fix the time and observe the height to which wetting took place in that time. It should be observed, that for a given time of partial immersion, the height of wetting is not necessarily directly proportional to the gain in weight since there may well exist a gradient of saturation extending from the face in contact with water to that portion of the specimen last wetted.

Table II gives data in part according to the method of measurement suggested by Washburn as previously quoted. The absence of several of the precautions and refinements suggested by Washburn will be noted. The most serious source of error is probably evaporation of water from the wetted brick. The investigator's eye was the chief dependence for controlling the depth of partial immersion. For those samples wetted through in 1 hour, data are provided for recording penetrability in terms of length of time required to reach a definite height. No measurements were made whereby penetrability could be estimated in terms of height for a fixed time.

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[1] See footnote 2, page 719.

[2] Research Associate for the National Terra Cotta Society at the U. S. Bureau of Standards, Washington, D. C.

[3] E. W. Washburn, "Porosity and the Mechanism of Absorption," *Journal*, Am. Ceramic Soc., Vol. 4, p. 920 (1921).

An examination of the data of Table II indicates that the wetting through of a brick, partially immersed, tends to give the same percentage of absorption without regard to the face through which the wetting takes place.

This is confirmed very nicely by sample No. 2*A* for all faces. Samples Nos. 8, 11, 12, 16*A* and 16*C* are in good agreement for the edge and flat faces, but not for the end face, wetting not having taken place from end to end during the hour. Samples Nos. 4*C*, 10, 26, and 27 are not in good agreement. It will be noted that the time of wetting for edge immersion was little short of the hour for these samples and as it is the time of first appearance of wetting on the top face that is recorded, the discrepancy may be explained by non-uniform travel through different portions of the brick. The theory of saturation gradients might also have a bearing on this lack of agreement.

For isotropic specimens, it would seem reasonable to assume that unit absorption would be constant without regard to the face through which wetting takes place when wetting through does not take place during a given time of contact.

Samples Nos. 6, 9, 13*A* and 13*B* which are the only soft mud bricks on which this relation can be tested gave values for unit absorption that are in passable agreement with this rule. Samples Nos. 14*A* and 14*B* gave high absorption on end as would be expected from their laminar end-cut structure. The side-cut bricks present some conformity to this rule. Samples Nos. 17*A*, 17*B*, 22*A*, 23*B*, 23*C* and 25 tend to approach the constancy postulated. In general the samples with low rates of absorption deviate widely from the expected constancy of unit absorption. Disregarding questions of non-isotropic structure and variable surface conditions, an explanation of the discordance of the low rate samples is provided by the fact that when actual gain in weight is small the errors of measurement (weighings) tend to become large percentage errors.

It should be noted that the volume-time rate of penetration and the height-time rate of penetration do not necessarily grade sets of specimens in the same order. For example, sample No. 3 (not included in Table II) is a very hard-burned dry-press brick with a water absorption of but 5.5 per cent by 5-hour boiling. By partial immersion flat it is wetted through in 20 minutes with an absorption of 3.3 per cent by weight at the end of the hour. Comparing sample No. 3 with sample No. 6 which is wetted through by flatwise immersion in 48 minutes, but has an absorption of 17.1 per cent at the end of 1 hour, it will be seen that sample No. 3 has the greater height-time rate of penetration but sample No. 6 shows a larger volume-time rate of penetration.

Table IV.—Time for Complete Absorption by Brick of One Milliliter of Water.

(All times in seconds.)

Sample	Number of Tests	Tested on End			Tested on Edge			Tested Flat		
		Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum
No. 1	3	12	15	10	7	10	5	8	10	6
No. 2A	25	5	13	3	4	13	3	4	8	3
No. 2B	6	4	5	4	4	4	4	4	4	4
No. 3	5	27	64	9	105	285	25	46	105	16
No. 4A	3	22	30	15	15	30	8	47	70	15
No. 4B	3	20	25	10	44	80	18	27	30	20
No. 4C	3	18	30	10	11	16	5	28	32	20
No. 5	5	16	45	6	47	150	12	24	70	8
No. 6	25	93	165	45	..	Over 300	45	54	90	20
No. 7	25	49	115	25	62	205	25	48	145	20
No. 13A	25	..	Over 300	52	..	Over 300	50	..	Over 300	53
No. 13B	6	48	70	30	87	120	45	30	70	10
No. 14A	25	32	130	10	...	Over 300	30	..	Over 300	20
No. 17A	25	Over 300	Over 300	Over 300	Over 300	Over 300	Over 300	..	Over 300	30
No. 17B	6	134	255	75	117	230	55	36	60	18
No. 19A	5	Over 300	Over 300	Over 300	Over 300	Over 300	Over 300	Over 300	Over 300	Over 300
No. 20	3	Over 300	Over 300	Over 300	Over 300	Over 300	Over 300	23	30	8
No. 23A	2	...	Over 300	150	Over 300	Over 300	Over 300	142	195	90
No. 23B	3	92	180	35	...	Over 300	60	32	55	20
No. 23C	3	136	220	35	...	Over 300	275	113	180	40

It is obvious that size, shape, number and direction of capillaries control the time-height relation. Considering the effect of variation in the diameter of the capillaries it is apparent that when the capillaries or openings are of sufficiently large diameter, the height-time penetration becomes zero due to the fact that the openings do not act as capillaries. On the other hand it is conceivable that very fine capillaries would reduce the height-time penetration due to viscous and frictional forces.

Table IV presents some data on penetrability as measured by the time required to absorb 1 ml. of water confined to an area bounded by a circle 2 cm. in diameter. It will be noted that the time of test was arbitrarily limited to 5 minutes.

The opinion is offered that this method, owing to variation in surface conditions of the specimens, is essentially qualitative. It serves to grade the samples into several classes quite sharply, but mathematical treatment of the results would most profitably be statistical in its nature. However, this method of testing has the very decided advantage of speed and ease of performance. It probably furnishes an estimate of what the brick mason calls "suction." Where, as in the case of sample No. 3, there is a decided discrepancy between the volume-time rate of penetration and the height-time rate, this method follows the height-time rating. An important distinction, however, is that the first two methods as used in this investigation operate against the force of gravity while gravity assists penetration in the present case.

This method of estimating penetrability was applied to some specimens of soft mud brick from Philadelphia which had formed part of a wall for over 150 years. It was found that the exposed faces of the specimens had been rendered practically impenetrable by the coating of tarry material from the city atmosphere. The unexposed faces had penetrabilities ranging between 12 and 50 seconds.

RELATION OF WATER ABSORPTION TO USES OF BRICK

Water Absorption as a Measure of Susceptibility to Disintegration by Weather:

There is undoubtedly a widespread belief among architects and engineers that the water absorption of a brick is a direct measure of its tendency to disintegrate under repeated freezings and thawings. It is believed that this opinion originates in the observation that for a given method of manufacture and for a particular clay or shale, the under-burned or salmon bricks have high absorptions compared with the well-burned bricks. In other words, this belief is sound if confined to one make of brick. However, consider a user of bricks whose experience has been confined to the district represented by samples Nos. 16*A*, 16*B* and 16*C*. In so far as he judges bricks by water absorption measured by 5-hour boiling, his experience will teach him that an absorption of 15 per cent or less will insure a brick of very satisfactory weathering properties. A 20-per-cent absorption represents a totally unsatisfactory brick for this district.

Until he learns better, this same user would probably accept bricks Nos. 17*B* and 24*B* with absorption of 12.9 and 13.2 per cent, respectively, under the impression that they were superior to the 15-per-cent absorption material previously found satisfactory. Actually samples Nos. 17*B* and 24*B* represent bricks which, in the opinion of the manufacturers, do not possess sufficient resistance to frost action to warrant use in the face of the wall. On the other hand our hypothetical user on the basis of his experience, would hesitate to use the brick of which sample No. 6 is typical (water absorption 22.9 per cent). Actually No. 6 has been used by the hundreds of millions for several generations past in a rather severe climate with satisfactory results. If our user still pins his faith to water absorption as a measure of the weathering qualities of brick he may now become conservative and demand that future selections of brick shall not exceed 3 or at most 5 per cent in water absorption. This does not solve the problem. Certain types of brick which fall within these limits for water absorption rapidly disintegrate by spalling owing to their structure. In other words water absorption takes its place with color and ring as a means of distinguishing an under-burned from a well-burned brick if and when the characteristics of the well-burned brick are known for comparison. Or, expressed another way, water absorption alone is not a measure of the weathering resistance of bricks in general, where differences in raw materials and methods of manufacture enter.

The above statement does not preclude the possibility that a measure of weather resistance may be derived from water absorption determined in conjunction with one or more other factors. Kreüger[1] in his paper on "Investigations of Climatic Actions on the Exterior of Buildings" reports that the ratio of apparent porosity to true porosity, called by him the coefficient of water saturation and hereinafter referred to as Kreüger's ratio, affords a partial means of differentiating between satisfactory and unsatisfactory weathing properties of bricks. The apparent porosity used in Kreüger's ratio is the cold absorption 24-hour partial immersion plus 72-hour total immersion expressed as a percentage of the bulk volume. The true porosity is the sum of all closed and open pores expressed as a percent of the bulk volume and calculated from the true specific gravity as determined on powdered material. Kreüger's theory which really dates back to Hirschwald[2] is, that as water expands 10 per cent in freezing, there must be at least ten per cent unfilled pore space present to provide the needed room, otherwise the brick will disintegrate. Kreüger considers "that a coefficient of water saturation higher than 0.90 is decidedly unsatisfactory and that the maximum figure should not exceed 0.85."

It is not evident from Kreüger's discussion just what the closed pores have to do with the case. It is possible that he dealt with material wherein the closed pores represented but a small percentage of the total porosity and the difference between his apparent porosity by cold absorption and the total porosity was largely due to "blind alley" pores. In any case it is considered that Schurecht's ratio (apparent porosity by 48-hour cold immersion to apparent porosity by 5-hour boiling) will serve as well as Kreüger's ratio, having the double advantage of not involving consideration of closed pores and of being much easier to determine experimentally. The last column of Table I gives Schurecht's ratio.

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[1] H. Kreüger, "Utredning rörande klimatisk inverkan på byggnadsfasader," *Ingeniors Ventenspakt Akademien No. 24*, pp. 68 to 81 and 92 to 103 (1923).

[2] J. Hirschwald, "Handbuch der Bautechnischen Gesteinsprüfung," Berlin (1912).

Before discussing the degree of correlation existing between Schurecht's ratio and resistance to freezing and thawing, a word is necessary as to rating of these samples from the standpoint of their weather resistance. No freezing-and-thawing tests were conducted on these samples. The manufacturer's classification was accepted in every case. It is assumed that where a sample represents the normal "well-burned" portion of the kiln and is sold and accepted as such for a period of years, then its degree of weathering resistance must be regarded as satisfactory for the particular climatic conditions to which it is exposed. However, the further consideration of differences in climate must be considered. A brick eminently satisfactory for southern Californian conditions might rapidly fail, when exposed to Great Lakes winters.

Table V.—Schurecht's Ratios for Well-burned and Under-burned Bricks.

Sample	Raw Material and Method of Manufacture	Schurecht's Ratio	
		Well-Burned	Under-Burned
No. 2	Dry press clay brick	0.74	0.92
No. 4	Semi-dry press clay brick	0.81	0.92
No. 13	Soft mud clay brick	0.72	0.99
No. 14	Stiff mud, end cut clay brick	0.72	0.91
No. 16	Stiff mud, end cut clay brick	0.78	0.87
No. 17	Stiff mud, side cut shale brick	0.71	0.85
No. 22	Stiff mud, side cut shale brick	0.77	0.80
No. 23	Stiff mud, side cut clay brick	0.765	0.89
No. 24	Stiff mud, side cut shale brick	0.61	0.80
No. 6	Soft mud, clay brick	0.90	....
No. 9	Soft mud, clay brick	0.88	....
No. 25	Stiff mud, side cut	0.88	....

While a carefully conducted series of laboratory freezing-and-thawing tests would undoubtedly be of help in the more exact rating of these specimens from the standpoint of resistance to disintegration by weathering, still it must be remembered that field observation is in itself the final standard for interpreting laboratory tests of this nature.

Considering the data given in Table I it will be observed that Schurecht's ratio is always higher for the under-burned bricks compared with the well-burned bricks of the same method of manufacture. However, as in the case of water absorption, the ratio for one manufacturer's salmon may be lower than the ratio for another manufacturer's well-burned bricks.

The validity of taking a definite value for Schurecht's ratio to serve as a dividing line between bricks of satisfactory and unsatisfactory weathering characteristics (well-burned and under-burned) can be tested by the data of Table V (arranged from Table I). Table V lists the Schurecht ratios of the samples which have been classified by the manufacturers as "well-burned" and "salmon." If 0.85 be taken as the value for Schurecht's ratio assumed to afford a separation between specimens having satisfactory and unsatisfactory weathering properties, it will be found that the value 0.85 does afford a good separation, according to the manufacturer's classification, until the stiff mud, side-cut shale bricks are reached. The salmon samples of manufacturer 22 and 24 give ratios of 0.80. On the other hand, samples Nos. 6, 9 and 25 offered as typical well-burned bricks of their district, suitable for exposure in the face of the wall, give ratios above 0.85. It may be remarked that one of these three is from southern California and its reputation might not be so excellent in another climate. However, the other two are from northern districts with notoriously variable winters. The opinion is offered that these ratios, while offering much better measures of weathering resistance than plain absorption, do not alone furnish a reliable guide. Still other factors must be considered. It is hoped that the mechanism involved in disintegration by climatic action will be made the subject of much future study.

Effect of Water Absorption on Water Transmission of Masonry. (Wet Walls):

Here, except for the work of Kreüger previously cited, we are in the realm of pure speculation, in so far as dependence is placed on laboratory controlled tests. Certain observations drawn from field experience are available and are herewith offered in the hope that they may profitably direct investigations.

As a result of observations in the field we can say that brick masonry, when intermittently wet on one face may or may not show wetting on the other face. The problem is to determine what the factors are governing the transmission of water through the wall when one face is intermittently wetted and dried. A probably incomplete list of these factors is given below, arranged without respect to probable importance:

1. Permeability of the brick units;
2. Permeability of the mortar;
3. Openings (cracks, holes) in the masonry joints (workmanship);
4. Volume-time penetrability of the bricks;
5. Height-time penetrability of the bricks;
6. Water absorption of the bricks;
7. Saturation gradients within the brick;
8. Effect of mortar joints on travel of water in wall;
9. Rate of drying of bricks from one face;
10. Effect of temperature differences on rate of drying and travel of water;
11. Effect of wind pressure on water penetration;
12. Nature of the construction.

Briefly discussing the above factors in order: The permeability of the brick unit has been little investigated. The data in the literature are in most cases for pressures enormously high compared to the pressures exerted by even very considerable wind velocities. It may be pertinent to note that Raisch[1] reports the permeability of brick per unit of area to *air* as $\frac{1}{380}$ of the permeability of the wall per unit of area built of the same make of brick.

The second factor, the permeability of the mortar, also has been measured at high pressures, but no data are available applying directly to the present problem.

The personal opinion is offered that the third factor is the one of outstanding importance in the list. This opinion has been formed after observations of wall constructions where the face of the wall was laid up with highly impervious and impenetrable bricks, a rich cement mortar used and a back-up of impervious units and in spite of all this the wall transmitted water abundantly with rains and winds of moderate duration and intensity. It was observed that the mortar joints had apparently shrunk, leaving cracks between the bricks and mortar, in some cases of such size as to permit the insertion of a knife blade an inch or more. The nature of the units being such that transmission of water through the units themselves did not appreciably take place, it would appear that this third factor remained by elimination.

The penetrability of the bricks, the fourth and fifth factors, would be expected to have an important bearing on wetting of the wall. Brick No. 2*A*, for example, would be expected to carry water through an 8-in. wall where laid as a header if the rain continued long enough.

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[1] E. Raisch, "Die Luftdurchlässigkeit von Baustoffen und Baukonstruktionsteilen," *Gesundheits-Ingenieur*, Vol. 51 (30), pp. 481–489 (1928).

However, if the water were limited to what would, under normal conditions, strike the end face during a rain, the time would be much in excess of the 28 minutes recorded for time of complete end wetting of this brick in consideration of the following. A precipitation in excess of 1 in. per hour is rare. Rain does not travel horizontally, hence, an actual contact of $\frac{1}{2}$ in. (1.25 cm.) per hour due to wind velocity would probably cover the actual rate at which water was supplied to the end of the brick. Three hundred milliliters are required to saturate brick *2A*. The end of this brick has an area of 54 sq. cm. Then 1.25 times 54 or 67.5 ml. is the amount of water available to be taken up per hour of rain. And 300 divided by 67.5 or 4.4 hours would be required to wet the brick through. This time represents a precipitation of 4.4 in. of water under the conditions stated. It should be pointed out that this calculation involves an assumption as to the effect or rather lack of effect of gravity on the saturation gradient in a brick concerning which there is not the slightest experimental evidence. The author is informed that it is the practice to plaster directly on 8-in. exterior walls built of the brick represented by *2A*. This would seem to indicate that high penetrability of the brick units is not in itself necessarily a cause of wet walls.

Brick No. *2A* is an extreme case of very high rate of penetration. It likewise represents a relatively high water absorption. From the viewpoint illustrated by the above calculation it would appear that the volume of water capable of being absorbed is an important factor in delaying striking through of moisture. Usually height-time penetrability and volume-time penetrability are parallel. Brick No. 3, which is an illustration of large height-time penetrability and very low volume-time penetrability, would be expected from this reasoning to transmit water quickly through the wall.

The fourth and fifth factors together with the sixth to tenth factors, inclusive, can be best handled by reviewing the experiments and opinions of Kreüger.[1] Based on inspection of over a hundred old and new buildings in various parts of Sweden and on a wide range of laboratory tests, Kreüger considers that "it is usually possible for a wall, not too thin, and not covered with too dense a surface layer, to dry sufficiently to prevent dampness from passing through to the inside." Kreüger's reasoning is as follows: Water will penetrate the face of the wall when the face of the wall is wetted. Once in the wall the water can escape in one of two ways. It can pass on through to the inside surface causing dampness or it can pass out through the face by evaporation during the intervals between rains. Experimental evidence is presented that the rate of evaporation per unit of surface is less for bricks of low porosity than it is for porous bricks. He strongly recommends use of porous material as a facing for the wall. Kreüger apparently regards materials having apparent porosities (four days, cold water) above 25 per cent as "porous." The corresponding water absorption by weight is roughly 14 per cent and above. Data are presented indicating that moisture moves away from the warmed face, that evaporation is greatly promoted by air currents, and that mortar joints interpose obstacles to the capillary motion of water from brick to brick.

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[1] H. Kreüger, "Utredning rörande klimatisk inverkan på byggnadsfasader," ***Ingeniors Venlenspake Akademien No. 24*** (1023).

He apparently pays little regard to the third factor though it is offered as the opinion of the author that the actual openings, very frequently present in the masonry, explain in part the noticeable effect of wind and rain combined in causing damp interiors.

Wind, as a factor in the water penetration of masonry (the eleventh factor), has been referred to above as effective in bringing rain to the surface of the masonry. Although wind pressures probably would have an important effect on the amount of water which would be transmitted through cracks or other relatively large openings, it seems doubtful whether such pressures of ordinary intensities would have an important effect on rate of water penetration through small pores of bricks.

It will be noted that the discussion under this section has been limited to consideration of water passing through a vertical wall section.

The twelfth factor (construction) is discussed by Palmer.[1]

Summary and Conclusions

This paper records the values for water absorption on 39 samples of brick representing several methods of manufacturing, and various degrees of burning. Water absorptions were recorded for 1, 5, 24 and 48 hours' cold total immersion, 5 hours' boiling and for 1-hour partial immersion on end, face and flat.

From these data on water absorption the following conclusions are believed justified:

1. When wetting through of the brick was completed during the time of partial immersion, the percentage absorption was in close agreement with the percentage absorption determined by total immersion for the same length of time.

2. After the brick was completely wetted, longer periods of immersion increased absorption by a much slower rate.

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[1] L. A. Palmer, "Wet Walls and Efflorescence," published by the American Face Brick Assn.

3. It is believed that the proposed relation, absorption by 5-hour cold immersion equals 75 per cent of absorption by 5-hour boiling, is not sufficiently accurate for specification use. If such a relation is required, more acceptable accuracy might be introduced by the use of a graded relation dependent upon the classification of the bricks.

Considering the data of other observers, it is apparent that 48-hour cold immersion did not give the maximum cold absorption possible with longer time of immersion.

Penetrability was measured at 1 hour as percentage gain in weight for partial immersion on end, face and flat. The time for wetting through was recorded when less than 1 hour.

From these data it was evident that the rating of penetrability was dependent upon the method of test used. The height-time rate of penetration and the volume-time rate of penetration are distinguished. The time for complete absorption for 1 ml. through a surface bounded by a circle 2 cm. in diameter was measured on certain samples.

On the basis of the data recorded in the present paper and field observations it is considered that the water absorption or apparent porosity of brick is not a measure of resistance to climatic action in so far as differences in raw materials and methods of manufacture are involved. However, for the same clay or shale and for the same method of forming, the difference between satisfactory and unsatisfactory weathering performance may usually be distinguished by difference in water absorption.

It is believed that while Schurecht's ratio (water absorption 48-hour cold immersion to water absorption, 5-hour boiling) is a much better measure of disintegration through freezing and thawing than is water absorption by any one method, it is not sufficiently reliable to furnish a basis for classification.

No laboratory data on transmission of water through masonry are available. However, on the basis of field observations compared with the known water absorption and penetrability of the units used, the opinion is offered that there was very little correlation between these properties and the transmission of water through masonry. Imperfectly closed mortar joints explained much of the observed results.

Acknowledgments.—The author acknowledges his indebtedness to the various members of the staff of the U. S. Bureau of Standards who gave assistance and counsel in the preparation of this paper. Particular thanks are due to Mr. D. E. Parsons for his suggestions relative to the mechanism of water transmission through masonry.

DISCUSSION

MR. T. C. TAYLER.[1]—During the past year I ran a few experiments on absorption of clay brick to determine the weathering qualities of brick of various degrees of absorption. I took from some buildings that were being torn down two brick adjacent to each other, one apparently a good brick and the other a badly disintegrated brick, and then tested them for percentage of absorption. I found that on certain makes of brick both good and bad samples showed high absorption, and on other makes, both the disintegrated sample and the sample that showed no disintegration had low absorption. I had about an equal number of brick of each class and each make. The brick that disintegrated did not have a high absorption, no higher than the brick which did not disintegrate.

The brick to which I refer was Detroit brick, a number of different makes, and most of them were rather old brick. My conclusion is that absorption is not a measure of the weathering qualities but that there is something else that is the measure, and perhaps it is the mortar in which the brick are laid. Perhaps it is the fact that where a brick is sealed in an almost watertight pocket that brick is continually maintained at its saturation point during freezing and thawing which causes it to suffer greater damage than some other brick where porous mortar permits drainage into the back-up materials. That is my conclusion.

MR. N. W. KELCH.[2]—I should like to ask Mr. Tayler one question about the brick that disintegrated alongside of those adjacent which, he said, did not. Do I understand that they were individual spotted disintegrated brick or was it a certain locality of a wall?

MR. TAYLER.—I took both kinds of cases. I had individual brick that showed disintegration and I had whole patches. Good and bad brick were taken from the wall immediately adjacent to each other.

MR. KELCH.—Did you have two bricks of a different characteristic of exposure or of the same exposure right adjacent to each other in the wall?

MR. TAYLER.—My idea is there must have been a difference in the exposure, in spite of the fact that good and bad brick were right adjacent to each other. However, when I took the brick out of a patch where the whole patch was showing disintegration, I sometimes had to go to the edge of the patch to try to get a good one which would be right next to one that had disintegrated.

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[1] President and General Manager, Sand Lime Products Co., Detroit, Mich.

[2] Secretary-Manager, Clay Products Institute of California, Los Angeles, Calif.

MR. L. B. LENT[1] (*presented in written form*).—This investigation by McBurney is a real attack on a problem which has vexed designers, contractors and owners for many years. Even though it may not be a complete solution, it is a start in the right direction and therefore a valuable contribution to our knowledge of both water penetration and durability. Eliminating possible causes enables us to concentrate our study on the lesser number of probable causes.

The thorough investigations discussed in this paper have exploded a widespread fallacy that the water absorption and penetrability of individual bricks is *the* important factor in water penetration of brick masonry and have thus furnished most convincing evidence of the correctness of the statement on this subject which Committee C-3 on Brick included in its annual report last year.[2]

To intelligently discuss the paper, one should have intimate knowlege of the technique of the tests which McBurney and others used, which I do not. But from some knowledge of these tests, and more as a result of field observation, I can agree with McBurney in his opinion that openings in masonry joints is the factor which most often governs the transmission of water through the wall, although the nature of the construction might be interpreted broadly to include some other named factors.

I have examined brick walls which were apparently soaked with water, the owner blaming this condition on the character of brick. By a casual inspection only, it was found that an open space nearly $\frac{1}{2}$ in. wide separated window frames from the adjoining brickwork, with no caulking or other mode of weather stopping. This is but one of the many instances noted where actual openings through at least the outer tier of a brick wall is surely the principal factor governing water penetration.

In some other cases, it was quite certain that poor mortar, that is, a weak mix, was passing water freely.

Regarding the possible disintegration of brick masonry, or permanence, as it is sometimes called, the reliability of Schurecht's ratio as a basis for brick classification, or indeed for a classification of other masonry materials, may still be found to have a useful application. I believe we may well continue our check investigations of this ratio against the known performance of masonry materials under a variety of long-time exposures and perhaps discover a closer agreement than was found in the relatively limited scope recorded in this paper. Perhaps the average will be found to be sufficiently high to warrant our use of this ratio in classifying brick for specification purposes.

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[1] Common Brick Manufacturers Assn. of America, Cleveland, Ohio.

[2] *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 306 (1928).

Laboratory investigations are perhaps most helpful when they have found practical application. And in this respect, McBurney's investigations are valuable because they confirm our belief that construction methods and practices are the more important considerations in preventing water penetration in masonry walls.

And finally, I think the conclusions might carry some emphasis on two important points, which I take the liberty of repeating in my own language, for they are often misunderstood and misapplied by some of our best-informed people. They are: (1) as it measures the ability of brick masonry to resist water penetration, the percentage of absorption of individual brick has no general application although it may have specific application in particular localities, and (2) as a measure of the durability or permanence of brick masonry, the percentage of absorption of individual brick alone is not even approximately accurate; although, in connection with other properties of the brick, it may be an important factor.

MR. D. K. BOYD.[1]—As an architect I wish to commend Mr. McBurney for this investigation, because it gives to our profession the beginning of the results which we may expect from the product, brick, as laid in the wall, in that it approaches the subject from the standpoint of absorption and saturation on one or two surfaces at a time and not by complete immersion or boiling, which, after all, gives more or less of an academic result. I should like to see this investigation continued further, if possible, by the testing of furred and unfurred walls consisting of bricks laid in different kinds of mortar with water applied on the exposed surface, just as a wall would be under actual conditions of construction and use in a building. Against such walls water could be played by a hose with a propeller, giving a wind motion, to simulate a rainfall with a certain wind velocity. These tests could be carried to the point where we might get what could be expected as the worst possible result in this climate of ours, namely, we might say, a three-day rainfall with a pretty stiff wind, and translate the whole thing into terms of what might be looked for in the actual wall which surrounds the occupants of a building.

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[1] Architect and Structural Standardist, Structural Service Bureau, Philadelphia, Pa.

MR. R. H. DANFORTH[1] (*presented in written form*).—This paper gives very voluminous tabulated information on a variety of tests of the water absorption of a large number of kinds of brick, as well as a very useful bibliography of recent related investigations both in the United States and abroad. The fact that so comprehensive a study as this leads to no more definite conclusions than those appearing in the summary indicates at a glance the difficulty of the problem which Mr. McBurney has attacked, and it is to be hoped that he will have the courage and pertinacity to continue his investigations along the lines he indicates.

It is unquestionably true that the water penetrability of a brick wall is affected by many other things besides the permeability of the brick, and that deterioration of brickwork by frost action frequently has little to do with the absorption of the brick units. In fact I almost doubt the possibility of specifying bricks and brickwork which will be satisfactorily impervious to severe storms until we have standardized the storms, as well as the bricks, mortar and workmanship.

As regards the deterioration of brickwork by frost action, my observations over thirty years in the climates of New England and the Great Lakes lead me to believe that by far the largest number of cases of such injury start at the mortar joints, and that spalling of the corners of the bricks is most frequently found after the pointing has worked out to such an extent that sleet can freeze between adjacent bricks, the resulting expansion having a tendency to chip off the corners of the brick. I hope others will be induced to give us their observations on this particular phase of the problem.

MR. S. H. INGBERG[2] (*presented in written form*).—The possibility that the unfilled but accessible pore space of a partially saturated burned-clay body affords information on resistance to frost action must be premised on the fact that the freezing of water in a wall proceeds gradually from the outer surface inward. The open ends of a water-filled capillary tube would apparently afford no relief if freezing occurred over its whole length at the same time. For gradual travel of the freezing zone, however, it appears reasonable that water inside of the zone can be forced into any unfilled pore space present by pressure of crystallization of water in the freezing zone. It is thus of much interest to note that the ratio of pore space filled by saturation methods comparable to what obtains for the outside of a wall to that filled by more nearly complete saturation has a bearing on weathering resistance. While this is one of the factors that may have to be considered it apparently does not obviate the necessity for considering other factors recognizable as important from the durability standpoint.

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[1] Professor of Mechanics, In Charge, Materials Testing Laboratory, Case School of Applied Science, Cleveland, Ohio.

[2] Chief, Fire Resistance Section, U. S. Bureau of Standards, Washington, D. C.

One point of interest is the relation the given ratio has to the total water absorption of the product. It appears both from the data presented by Kreüger[1] and by the present author that this ratio generally decreases with decrease in absorption. In the accompanying Table I, the data from the author's Table I has been placed in four groups determined by respective absorptions ranges of below 6 per cent, 6 to 12 per cent, 12 to 20 per cent, and above 20 per cent. While considerable variation obtains as concerns maximum and minimum values, the average values for Schurecht's ratio are seen to increase fairly consistently with the percentage absorption for the corresponding group averages.

TABLE I.—RELATION OF TOTAL ABSORPTION BY 5-HOUR BOILING WITH RATIO OF ABSORPTION BY 48-HOUR IMMERSION AT ROOM TEMPERATURE TO THAT BY 5-HOUR BOILING.

(Data from author's Table I)

Number of Samples in Group	Absorption by 5-Hour Boiling, per cent			Ratio, Absorption by 48-Hour Immersion to Absorption by 5-Hour Boiling		
	Minimum	Maximum	Average	Minimum	Maximum	Average
4	4.9	5.6	5.1	0.51	0.83	0.64
12	7.6	11.5	9.3	0.51	0.81	0.74
15	12.5	19.2	15.7	0.62	0.99	0.80
8	20.7	27.6	23.8	0.74	0.92	0.85

Other considerations indicate that conditions of full saturation must obtain in portions of a brick or group of bricks, for which the average saturation may be only partial. Thus, if an average saturation as by 48-hour immersion of 80 per cent obtains, this does not mean that all channels and portions of the bricks are uniformly filled to this extent with a connected volume of pore space of 20 per cent that is not filled. Instead, we must consider the 80 per cent as representing only the average for all of the channels, parts of the unit or units, that some are fully filled and others by more or less than 80 per cent. That such variability holds for burned-clay products such as bricks is indicated by the range between maximum and minimum values obtaining for a sample of only five units. On both absorption and strength, variability of 25 per cent or more above and below the average value frequently obtains and this is recognized in individual maximum and minimum values prescribed in specifications. While percentage saturation is a relative quantity and thus can be expected to vary less than percentage absorption, the possibility of a large range in nominally comparable bricks is apparent, particularly if extended to include all exposed bricks in the walls of a building. From the standpoint of weathering resistance failure of one brick in one hundred or several hundred cannot be considered satisfactory performance.

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[1] *Ingeniors Ventenspake Akademien No. 24* (1923).

The variation obtaining can be pictured, perhaps somewhat conservatively, as in the accompanying Fig. 1 for an average saturation

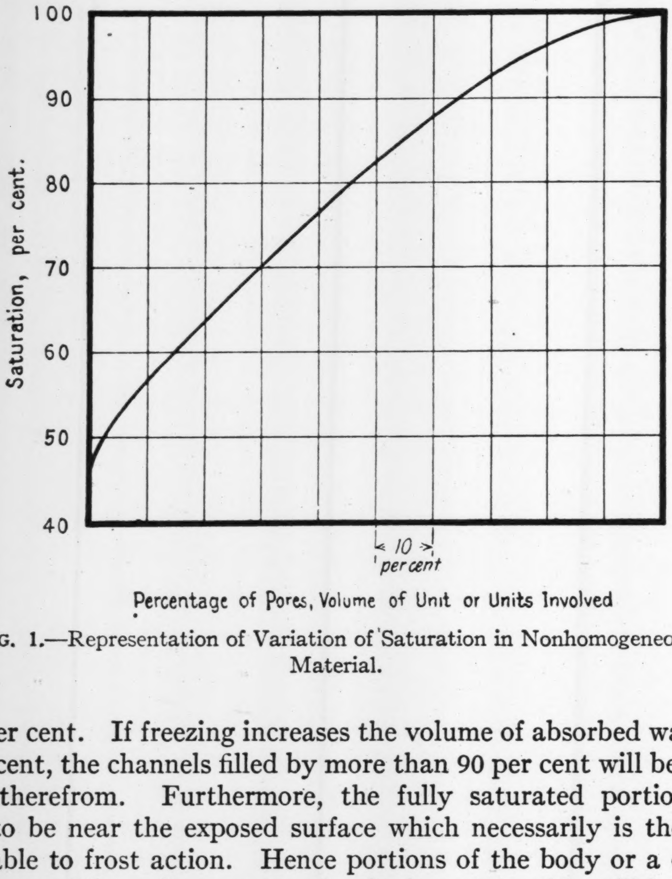

FIG. 1.—Representation of Variation of Saturation in Nonhomogeneous Material.

of 80 per cent. If freezing increases the volume of absorbed water by 10 per cent, the channels filled by more than 90 per cent will be under strain therefrom. Furthermore, the fully saturated portions are likely to be near the exposed surface which necessarily is the most vulnerable to frost action. Hence portions of the body or a certain percentage of the units or parts of units in an assembly will be under conditions approximating full saturation, where only partial saturation obtains for the body or assembly as a whole, with less relief from the expanding forces of ice formation than is assumed to obtain for the partially filled channels.

Hence the total water content of a body and the properties that enable it to resist frost action under conditions approximating full saturation also appear important. Resistance factors of the latter type include cohesiveness, expressed perhaps as tensile resistance, and and absence of planes of weakness. These are in part functions of the degree of maturity to which the product is burned, the importance of which in relation to weathering resistance is fully recognized in the paper.

MR. D. W. KESSLER[1] (*by letter*).—Evidently one of the main intentions of this paper was to discuss the relation between water absorption and weather resistance, and it was pointed out that absorption, in general, is not a reliable indicator of durability. If the durability of different masonry materials were inversely proportional to absorption, one could easily compute the relative serviceability of various types, or by determining the average life of one type, that of the others could readily be determined. One might, for instance, assume that a brick having an absorption of 10 per cent would last 50 years, then if our absorption theory were valid the life of other materials would be roughly as follows:

$$\begin{aligned}
\text{Limestone} &= \frac{50 \times 10}{5} = 100 \text{ years} \\
\text{Granite} &= \frac{50 \times 10}{0.3} = 1666 \text{ ''} \\
\text{Marble} &= \frac{50 \times 10}{0.1} = 5000 \text{ ''} \\
\text{Slate} &= \frac{50 \times 10}{0.2} = 2500 \text{ ''} \\
\text{Concrete} &= \frac{50 \times 10}{8} = 62 \text{ ''} \\
\text{Terra Cotta} &= \frac{50 \times 10}{10} = 50 \text{ ''} \\
\text{Cast stone} &= \frac{50 \times 10}{7} = 71 \text{ ''}
\end{aligned}$$

Such a computation illustrates the fallacy of placing too much dependence on the absorption test. Experience has proved that the life of marble masonry in this country is nearer 100 years than 5000. A slate roof should last 100 years, but instances are known where slate roofs have failed in less than 50, while granite is usually considered our most durable material.

The weathering of masonry involves so many different factors that no single test can ever be of much value. In this country we have been too apt to attribute all cases of decay to frost action. While frost may strike the vital spot of some materials it seems very doubtful if it is an important factor in the decay of those materials of very low absorption.

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[1] Civil Engineer, U. S. Bureau of Standards, Washington, D. C.

Absorption "by weight" as reported in this paper may be misleading when materials of quite different bulk weights are compared. For instance, brick *2A* has a bulk weight of 1.57 g. per cu. cm. while that of brick No. 21 is 2.36. From the results given one might be led to believe that brick *2A* absorbs nearly 4½ times as much as No. 21. When reduced to volume absorption percentages it appears that the actual absorption of brick *2A* is just about twice that of No. 21. The weight ratio is so simple to determine that it will probably always be used, but in a paper of this kind it seems advisable to give volume ratios as well.

MR. J. W. MCBURNEY.[1]—Referring to the remarks by Mr. Tayler, unfortunately the local clay brick made in Detroit happens to be one of the exceptions to the general rule that under-burned bricks have a higher absorption than the well-burned bricks made from the same clay or shale. Some salmon bricks from the Detroit district were recently examined and it was a matter of some astonishment to find that the water absorption of these under-burned or salmon bricks was several per cent lower than that of the otherwise comparable, normal or well-burned bricks. Schurecht's ratio, however, was close to unity for these salmon specimens. I agree with Mr. Tayler that the difference between satisfactory performance and failure of these units might be largely determined by the conditions in the wall under which a given brick did or did not approach complete saturation with water.

Referring to Mr. Ingberg's remarks I would point out, with reference to his table, as I said in another paper, I am afraid of averages. I gave as a facetious example the illustration of a field occupied by an equal number of cattle and horses concerning which it was hardly proper to say that the animals had on an average one horn apiece, and that is sometimes what we get into when we are dealing with averages which include entirely separate categories.

I am thinking especially of Mr. Ingberg's comparison where he averages eight makes of brick with water absorption above 20 per cent, getting an average of 23.8 per cent absorption by boiling and 0.85 for the average of Schurecht's ratio. According to my record Nos. *2B*, *14B* and *16C* included in this average are under-burned brick.

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[1] Research Associate of the Common Brick Manufacturers' Assn. of America **at the** U. S. Bureau of Standards, Washington, D. C.

The average water absorption by boiling of these three is 23.8 per cent and the average of their Schurecht's ratio is 0.90. For the remaining five makes, the water absorption averages 24.3 and the Schurecht's ratio is 0.82. It should be pointed out that certain of these "well-burned" samples are from Southern California. Hence whether they can be considered really frost resistant is an open question. In any case it seems that if variation in Schurecht's ratio is to be regarded as a measure of resistance to freezing and thawing it will not add to our knowledge to consider average ratios where both well burned and under-burned specimens are averaged together.

Professor Kreüger of Stockholm indicated in a communication to the author that according to his experience bricks contain no closed pores worth mentioning, and from this consideration, he considers that the tests indicate the reliability of his ratio. He is doubtful of the utility of boiling tests.

SYMPOSIUM ON MINERAL AGGREGATES

INTRODUCTION

By R. W. Crum[1]

Specifications for aggregates, methods of use, and methods for inspection are very largely local problems, and must usually be solved from a local point of view. Engineers in one section of the country may make successful use of aggregates that it would not be good policy to use at all in some other section. It is doubtful whether it will ever be possible to devise standard specifications for aggregates, that will be anything more than a form upon which to hang the requirements best adapted to the different localities.

The primary work of this Society and others with respect to aggregates is to develop and make available a mass of sound information concerning the effects of different characteristics of the materials under the various conditions of use, so that the local user can specify test limits for his particular purpose in an intelligent way.

It will become evident from the papers presented at this Symposium that research in this field has only been started and that much information is needed to give the users the needed assistance. It will also be apparent that owing to the wide range in characteristics and conditions of use, thorough correlation of the work of the various research agencies and of their results will be needed before sound recommendations can be made.

The papers presented at this symposium have been secured through a committee, advisory to the Society's Committee E-6 on Papers and Publications, consisting of the following:

Julius Adler
A. T. Goldbeck
F. H. Jackson
H. J. Love
W. E. Rosengarten
Stanton Walker
R. W. Crum, chairman

These papers represent the individual views of their authors, the function of the committee having merely been to arrange for the

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[1] Director, Highway Research Board, National Research Council, Washington, D. C.

presentation of the various phases of the subject by men qualified to speak with authority.

By way of introduction I wish to comment upon some factors connected with the inspection of aggregates, which are not otherwise covered.

ORGANIZATION FOR INSPECTION

Aggregates are among the most difficult of all materials to inspect fairly. It is easy of course to write specifications for ideal material and then accept only shipments in perfect compliance therewith, but it is better engineering and better economy to devise methods for using the general run of aggregates available in the locality, and then inspect shipments in a reasonable way so as to be fair at all times to both producer and consumer.

In organizing for the inspection of aggregates the first thing to bear in mind is that the rights of all parties to the contract must be scrupulously maintained.

It is the right of the consumer that each unit of material offered for inspection shall conform to the specifications agreed upon in the contract.

It is the right of the producer that the inspection methods shall determine the true facts with reference to each unit of inspection.

Most construction contracts provide that no matter what inspection takes place during the progress of the work, the contractor is not relieved of his responsibility for producing completed work in full accordance with the specifications. Nevertheless I have long believed that in the case of aggregates the inspecting agency cannot deny its responsibility for making full acceptance when the official inspection is made, excepting, of course, in case of fraud. Cases sometimes arise in which aggregates accepted by an official inspector are later found for one reason or another to be defective. Rejection of this material often involves a considerable loss, and fairness to the producer requires that the responsibility for the mistake be determined by the highest authority available. Such cases should be adjudicated by the engineer in charge of the work or at least by his immediate representative. If the material was loaded during the usual routine operations of the plant without unusual circumstances and the error was that of the inspector in failing to note the objectionable features, the consumer should pay for the material and the transportation charges and dispose of the material as best he can. The fate of the inspector is another matter. On the other hand if the evidence shows a deliberate attempt to deceive the inspector or to take advantage of him on a busy day, the material should be summarily rejected. These points are difficult of determination and should be considered in a judicial spirit.

Aggregates are likely to be variable as they come from the pit, quarry or stock pile, and the determination of the true quality of a shipment calls for great care in sampling and the use of good judgment in interpreting the test results. Although specifications usually provide for decision on the basis of the tests of the samples taken as specified, the producer should in fairness be given as many tests as are necessary to establish the true characteristics of the unit inspected. The testing of aggregates is not so exact that one can well refuse to check his work.

Aggregates being what they are, there are often variations from one part of the unit to another. A sample secured in one way may fail and another may pass. If the average is below specification requirements there is no warrant for accepting the material, but if the average is good a doubt sometimes arises. The inspector is then faced with the necessity of deciding whether the material will be sufficiently mixed in handling before use so that the average will govern, or whether it will be used in such a way that the low grade portion will go directly into the structure. This requires that a competent inspector be familiar with the conditions under which the material is to be used and that he understand the effects of these conditions upon the final result.

INSPECTION TOLERANCES

In the inspection of aggregates, some degree of tolerance on the test limits is usually necessary. There are two sources of variation that should be considered in setting tolerance limits. There are some variations due to irregularities in the natural deposits which are out of the control of the producers, such as shale and soft particles in gravel pits and soft stone layers and earthy intrusions in quarries. When objectionable characteristics show up that cannot be corrected in the plant the producer's only recourse is to divert the material to other uses or move to another part of the pit or quarry. In such cases the tolerance limit should be as liberal as is consistent with the intent of the specifications. Preferably such tolerance should be established to meet the individual characteristics of the various sources of supply. Other variations depend upon the plant processes, such as grading, dust or silt content, etc. The tolerances for such controllable factors should be as close as can be maintained at reasonable cost and should be rigidly enforced.

SIZE OF INSPECTION UNITS

Inspection units vary in size from a wheelbarrow load to an entire source of supply. Probably the most common unit is the car load. The permissible maximum size of the unit perhaps ought to depend upon the uniformity of the product, but the inspection is usually limited to the cars as loaded. If the material is not remarkably uniform, arrangement should be made for sampling from the chutes as the car is loaded, for it is exceedingly difficult to get samples from a loaded car that will represent a fair average. This matter is also closely related to that of the place for inspection.

PLACE FOR INSPECTION

There is a difference of opinion as to whether aggregates should be inspected at the producing plant or at the site of the work. There are notable examples of large organizations using either practice, and no doubt the real truth of the matter is that both are adequate and successful for the conditions under which they are operating. Mr. Rea has given a most informative discussion of this topic in his paper for this symposium.[1] Fundamentally the system that should be used depends upon justice to the contracting parties, uniformity of products, and economy.

If the natural material is uniform in quality and the only source of non-uniformity is in the plant preparation of the product, it may be entirely satisfactory to inspect upon delivery. This will certainly entail the least expense to the consumer. In such a case rejections will be very infrequent, and evidences of careless plant operation can usually be noted in time to take corrective measures before real trouble results.

On the other hand if the natural supply is variable in quality it will generally be much more satisfactory and economical to inspect the aggregates at the producing plant so that rejections will not involve loss of time at the job and loss of the transportation charges. If the producer is faced with frequent costly rejections after shipping the material to the site of the work, the consumer may expect to bear the extra cost in the higher price level that will prevail. Of course the producer may safeguard himself by providing his own inspection, but this adds to the cost of production, to be borne ultimately by the buyer, and still does not carry complete protection as the buyer's inspector has the last word. In such situations the consumer should assume the obligation to pass upon the material before haulage charges accrue.

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[1] A. S. Rea, "Methods of Inspection of Mineral Aggregates," see p. 745

INSPECTORS

Inspectors of aggregates should be mature men who have an intelligent conception of the use to which the material they inspect is to be put, and who have some knowledge of the effects of the characteristics of the material upon the resulting product. They should have authority to make decisions that will be respected. Men are needed in whom their superiors may have confidence that their judgments will be correct. Such men are entitled to the backing of their superiors.

METHODS OF INSPECTION OF MINERAL AGGREGATES

By A. S. Rea[1]

Synopsis

This paper discusses the methods of inspection of mineral aggregates from a technical standpoint. The discussion considers: (1) preliminary investigations, including material surveys and reconnoissance surveys; (2) inspection during construction, including inspection at source of supply and at destination; (3) methods of sampling; and (4) field tests.

The important things to be observed in inspection at the source of supply and at destination are pointed out and the advantages of each method of inspection are discussed. The importance of proper sampling is emphasized and the standard methods of the Society are recommended as a guide in formulating instructions to inspectors. The screen test, silt test, colorimetric test for sand and test for weight per cubic foot are named as the more important field tests.

Mineral aggregates play a most important part in the construction work throughout the country. Our highways and streets, for example, are over 90 per cent mineral aggregates and concrete construction of all kinds will average 85 per cent aggregates. Therefore adequate inspection and tests are necessary to the protection of the enormous annual investment in these materials.

Assuming that provision has been made in the matter of organization by the user, whether it be a unit of national, state, county, or municipal government, or private organization to handle inspection of materials, certain fundamental rules can be laid down in the methods to be followed. With the varying conditions existing both as to locality and types of construction it is recognized that it is impracticable to formulate specific directions which would be applicable generally. For this reason no attempt will be made to enumerate in detail, instructions to be followed in the inspection and sampling of materials. The subject will be discussed for the most part by outlining in more or less general terms the methods and policies which have been used successfully by engineering organizations, and references will be made to standard methods or procedures that have been adopted.

The subject will be considered under the following sub-titles:

1. Preliminary Investigations;
2. Inspection During Construction;
3. Methods of Sampling; and
4. Field Tests.

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[1] Chief Engineer, Bureau of Tests, Ohio Department of Highways, Columbus, Ohio.

PRELIMINARY INVESTIGATIONS

Preliminary investigations of aggregates may be said to cover any examination or inspection of the source of supply prior to construction. In practice this may consist of either geological or material surveys, or reconnoissance or special preliminary examinations.

Material Surveys:

A number of the states as well as other political sub-divisions of government have had very complete surveys made of their local resources. This has been especially true with reference to deposits of mineral aggregates. In some cases the survey has been conducted under the supervision of the state geologist, while in others it has been under the direction of the highway department. The information collected in a survey of this sort may be of inestimable value in planning and carrying out a construction program. A materials survey, involving the sampling and testing of the materials, is considered of such value that it is essential that the investigation should be in charge of someone who not only knows the geology of the state, but is also familiar with the character and use of construction materials.

Some of the important things to observe in a survey of this kind include the following: a detailed description of the deposit or quarry including the exact location; name of owner; the geological formation where applicable; the amount and character of over-burden or stripping; approximate quantity available; whether material from the same source has been previously used, where and for what purpose, and with what results. Notation should also be made of the shipping facilities, the plant equipment, if it is a commercial plant, and the approximate capacity of the plant. A detailed record of the extent and location of the material represented by each sample should be made. For this purpose a sketch, both in plan and elevation, showing the thickness and location of the different layers is recommended.

Special Preliminary Surveys:

In those states where no general materials survey has been made and even in those where such a survey has been conducted at a much earlier date, where possibly conditions have materially changed since the original survey, it is often desirable to make a reconnoissance or special materials survey. The plan usually followed when a construction project is proposed is to have an examination or survey made to determine what local materials are available and also their suitability for the particular type of construction contemplated. In some states, where conditions are such that it is considered impracticable to ship in material for particular projects, the specific requirements for the mineral aggregates are based on the test results of the best material available locally. A careful study should be made by the engineer of all surface indications and exposures and a sketch or map should be made of the project and adjacent area, from which it may be determined whether it is more economical to haul from the local deposit than to ship material from a commercial plant by rail. The procedure to be followed and the data which should be collected are essentially the same as that noted under "Material Surveys," except that perhaps less detailed information will usually be required.

INSPECTION DURING CONSTRUCTION

Assuming that the contract has been awarded and that the contractor has advised the materials engineer as to the source from which he proposes to obtain his mineral aggregates, the method of handling the routine inspection may be either by inspection at the source of supply or by inspection at destination.

An inquiry as to the methods followed by various states, counties, cities and organizations in this respect will reveal a marked difference in policy in the matter of inspection. Some states, for example, strongly favor the inspection of all aggregates at their source. In others the tendency is for inspection at destination, while in most states a combination of the two methods in varying ratios is the policy. Conditions, of course, determine more or less the practicability of the best method to follow. The chief advantage in the inspection at the source of supply of any material is in avoiding delay awaiting the results of laboratory tests. It also practically eliminates the possibility of rejection on the job with the resulting expense and delays involving re-shipments, demurrage, etc. The chief objection to this method of those favoring inspection at destination is based on the high inspection charges where the shipments are light, together with the fact that where conditions are favorable the plant inspection as applied to aggregates is considered unnecessary. It is held that where the quality of the aggregate is not in question and the plant operation is efficient, so that the grading or sizing is properly controlled, it is a needless expense to provide for plant inspection under such conditions. Where the quality and the grading as evidenced by the character of the product received is not satisfactory, plant inspection may be desirable and economical. In this connection it might be said that the more general adoption of field tests, as applied to aggregates, has resulted in the elimination of many delays in awaiting the results of tests on samples sent to a central laboratory.

Inspection at Source of Supply:

It is not the intention to enumerate the duties of a plant inspector nor to go into detail regarding the inspection methods to be followed for each type or class of fine and coarse aggregate, but rather to outline the general practice normally followed at plants producing aggregates for construction work and to call attention to some of the precautions to be observed.

The materials inspector is stationed at the plant for one purpose—to see that material being shipped complies with the requirements of the specifications and is satisfactory in every way. It is his duty to see that material being loaded is of approved quality and of satisfactory grading and uniformity, and in some cases to keep a record of the car weights. Inspectors located at plants furnishing fine and coarse aggregates should make a careful study of plant conditions and operations, with a view to being able to make suggestions on improving the quality or uniformity of the product. In taking samples of aggregate, whether they are for his own field tests or are to be sent to the testing laboratory, he should keep in mind at all times that the one fundamental requisite is that the sample must be representative of the material being sampled. It is often a very difficult matter for the inspector or engineer on the job to secure a strictly representative sample from a car or stockpile, particularly of coarse aggregate. The inspector at the plant has a much better opportunity to obtain proper samples while the material is being loaded into cars or bins.

One of the very important things for an inspector to observe at a plant producing gravel, crushed stone or slag is the size and uniformity of grading. Some of the factors affecting the size of coarse aggregate at the plant are: size and shape of screen openings; length of screen sections; the angle of screen with horizontal; the speed of screen rotation; and the rate at which the screen is fed. A general inspection and record of these conditions should be made by the inspector.

Inspectors at plants producing aggregates should see that coal, cinders, and other foreign materials are removed from cars or barges before the aggregate is loaded for shipment. When the calking of car bottoms is necessary, the inspector should see that it is done in such a manner and with such material that the aggregates will not become contaminated with foreign material during the unloading operation. The use of boards, roofing paper or burlap has been found to be the most satisfactory for this purpose and their use should be encouraged by the inspector.

One of the important duties of the plant inspector is to see that accurate records are kept of all shipments and test results. He should also see that all samples sent to the laboratory are properly marked and identified.

Inspection at Destination:

Considering the construction and building program as a whole it is probably a conservative guess that over 90 per cent of the inspection of aggregates is at destination. While a few engineering organizations provide for plant inspection for practically all of their construction work, most of the states, counties, cities and private organizations provide only for the inspection and testing of the aggregates after delivery. The ratio of plant inspection to inspection at destination as applied to other construction materials, particularly manufactured materials such as portland cement and steel, would be much higher than with aggregates. This is due to the fact that the inspection of such materials as sand, gravel, stone and slag lends itself better to visual inspection and also perhaps to the fact that field tests are more adaptable to aggregates than to other classes of materials.

It has been found desirable by some organizations to handle the inspection of all materials at destination by specially trained materials inspectors. In most instances, however, the inspection of materials is supposed to be taken care of by the same engineer or inspector in charge of the construction work. Unfortunately the inspection of the materials has been in many cases sadly neglected. Too much dependence has been put upon the ability to judge the quality by visual examination. Conditions, however, are improving each year and more attention is given to the inspection and testing of materials.

The inspector or engineer responsible for the inspection of materials at destination should examine all shipments of fine and coarse aggregate upon arrival to determine so far as possible from an examination of the top of the car, if the material complies with the general requirements of the specifications relative to uniformity, cleanliness and freedom from foreign or objectionable material. Whereas the inspector at the plant has the opportunity to thoroughly inspect and sample the car as it is being loaded and is thus in a position to judge the quality of the entire shipment, the inspector on the job has much more of a problem in the inspection and sampling of the car prior to unloading.

Providing the quality and grading of the material are satisfactory so far as is possible to determine from a visual examination of the car, the next step is to take samples for field or laboratory tests. When samples are to be taken prior to the unloading of the car, special efforts should be made to secure as nearly representative samples as possible. Samples should be taken at a number of points including the middle and both ends of the car by digging holes at least two or three feet deep and starting at the bottom of each hole, drag the point of the shovel up the side. The samples thus taken from each hole should be mixed and the composite sample quartered down to the desired size.

While ordinarily samples of each shipment are taken for either field test or laboratory test, it may be that at times the aggregate will be of such a character that the car may and should be rejected outright on the basis of visual examination alone; for example, a car of sand may be excessively dirty or badly contaminated with foreign material. Coarse aggregate for concrete, such as gravel, may be badly coated or contain clay lumps. Crushed aggregate, such as stone or slag, may obviously be of improper grading or may be exceedingly non-uniform, containing perhaps pockets of fine material or dust in the center and excess coarse material at the ends. In cases of doubt the material should be subjected to the field tests on the job or samples sent to the laboratory, but there are many instances in which the quality or grading is obviously so far at variance from the specifications that the inspector may be fully justified in rejecting the material without test.

METHODS OF SAMPLING

The methods of sampling are considered separately from the inspection, for the reason that the sampling of materials is required under both material surveys and preliminary surveys as well as under routine inspection, whether at the plant or at destination. It is not considered necessary to go into detail as to just how the samples shall be taken or the quantity of sample required for each class of fine and coarse aggregate, for the reason that the methods of sampling aggregates are covered in one of the standard methods of the Society.[1] This method has been approved as "American Tentative Standard" by the American Standards Association and also with only slight modifications as a "Tentative Standard" of the American Association of State Highway Officials.

It is believed that experience has demonstrated that the methods of sampling prescribed in this standard have proved very satisfactory and practical and that any engineering organization formulating instructions to inspectors would do well to refer to this standard as the basis for its sampling requirements.

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[1] Standard Methods of Sampling Stone, Slag, Gravel, Sand and Stone Block for Use as Highway Materials, Including Some Material Survey Methods (D 75 – 22), 1927 Book of A.S.T.M. Standards, Part II, p. 473.

FIELD TESTS

A discussion of methods of inspection would not be complete without reference to some of the more important "field tests" which are now in more or less general use in connection with the inspection of aggregates.

Field tests may be made at the source of supply or at destination. The most important of the tests in the field on fine and coarse aggregate are the following:

1. Screen Tests;
2. Silt Test;
3. Colorimetric Test for Sand; and
4. Weight per Cubic Foot.

Screen Tests:

The field test in most general use is the grading or screen test of the aggregates. In many instances, particularly with crushed rock, the material has been subjected to complete laboratory tests for quality, so that it is only a matter of determining whether the aggregate is of satisfactory grading and uniformity. For this purpose the inspectors are furnished with screens and sieves, and either scales for determining the proportion of the various sizes by weight, or suitable cylinders or containers for volumetric determinations. The grading of an aggregate can be determined in the field practically as accurately as in the laboratory; therefore, except in very extreme cases, it should not be necessary to hold up construction work pending the results of laboratory tests when only the size is in question.

Silt Test:

The percentage of silt or clay in a sample of sand may likewise be determined in the field by a proper washing or elutriation test. The most accurate and satisfactory method is by the weight method, in which the silt is washed out in a specified manner and the loss in weight determined as described in the Society's Standard Method of Decantation Test for Sand and Other Fine Aggregates (D 136 – 28)[1] This method requires a source of heat for drying out the sample prior to and subsequent to the washing. A more rapid and simpler method sometimes employed as a field test consists in shaking a sample of sand with water, in a glass jar or cylinder, after which it is allowed to stand for a stated length of time and the percentage of clay or silt settling out on top of the sand estimated. The value obtained by this method is usually from two to four times that obtained by the weight method, depending largely upon the time allowed to settle; hence this method is only considered as a guide and may in cases of doubt necessitate a check determination by the laboratory.

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[1] 1928 Supplement to Book of A.S.T.M. Standards, p. 161.

Colorimetric Test:

The colorimetric test for organic impurities in sand is one of the field tests which is now being used quite extensively. The test consists, briefly, in adding a 3-per-cent solution of sodium hydroxide to a given volume of sand in a graduated bottle and noting the color of the solution after 24 hours. If the liquid is colorless or has a slight yellow color the sand may be considered satisfactory in so far as organic impurities are concerned. The method is described in the Society's Standard Method of Test for Organic Impurities in Sands for Concrete (C 40 – 27).[1] The field method is the same as the laboratory method described in the standards, except that a color chart is used for comparison instead of employing colored solutions prescribed for the laboratory method.

Weight per Cubic Foot:

One of the usual requirements in specifications for certain classes of coarse aggregate such as blast-furnace slag is a weight per cubic foot requirement. On inspections covering this type of aggregate, therefore, it may be required that this determination be made in the field. A determination of the unit weight of other coarse aggregates is also sometimes necessary in connection with weight proportion of aggregates for concrete and also in connection with designed mixes, where proportions are based on compacted weight of aggregates. A description of the method appears in the Society's Standard Method of Test for Unit Weight of Aggregate for Concrete (C 29 – 27).[2]

Those cited are the more important and generally used field tests in connection with the inspection of aggregates. Other special field tests employed where conditions require include such tests as a determination of the percentage of shale in gravel; the percentage of coke or coal; the percentage of clay lumps; the percentage of soft fragments; and the percentage of thin or elongated pieces. In some cases also where a crushed gravel is specified, it may require a field determination of the percentage of angular pieces in the aggregate.

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[1] 1927 Book of A.S.T.M. Standards, Part II, p. 123.

[2] *Ibid.*, p. 120.

SUMMARY

It has been estimated, it is believed conservatively, that of the billions of dollars expended in this country annually in construction work, not less than fifty per cent is represented by the cost of materials incorporated in the work. Mineral aggregates play a most important part in this construction program, exceeding, from the standpoint of volume, any other class of material used in construction work. Our highways and streets to the extent of over 90 per cent are composed of mineral aggregates. Concrete construction of all kinds will average approximately 85 per cent by volume of mineral aggregates.

It goes without saying that the protection of the enormous investment represented by this construction program makes mandatory the assurance that proper selection of materials be made. This can only be done by adequate inspection and test methods. This paper has attempted to outline an efficient and practical procedure to follow in the inspection of mineral aggregates, both at their source and at destination.

FINE AGGREGATE IN CONCRETE

BY H. F. GONNERMAN[1]

SYNOPSIS

This paper gives a general discussion of the rôle of fine aggregate in concrete, and includes brief statements from the published writings of some of the early writers on the subject, who were concerned with many of the problems that are discussed in current engineering literature.

The function of fine aggregates in concrete and the principal considerations involved in their selection are discussed in detail. The desirable characteristics of a sand for use in concrete are stressed, including durability and cleanness of the particles and tests for these properties are reviewed. The use of crushed stone or air-cooled slag screenings as fine aggregate is also discussed.

The influence of size and grading of fine aggregate on such properties as strength, permeability, and workability of concrete is pointed out, and the principal requirements of typical specifications for fine aggregate summarized.

INTRODUCTION

Of the four ingredients—cement, water, fine aggregate, and coarse aggregate—which normally enter into the manufacture of concrete, the one which has probably received more attention from investigators than any of the others is the fine aggregate. This, no doubt, arises from the fact that in the preparation of mortars and concretes from hydraulic as well as non-hydraulic cementing materials, it was early recognized that the characteristics of the fine aggregate had a very important effect on workability, strength, uniformity, and other properties of the resulting product. A review of the published literature on this subject shows that the early writers were concerned with many of the same problems that are with us to-day, and were acquainted with some of the fundamental factors affecting quality of mortar and concrete. In this connection the following excerpts from a number of published treatises are of interest.

Vitruvius, the great Roman architect, writing during the time of Augustus Caesar (about 25 B. C.) made the following statements concerning sands for mortar:[2]

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[1] Manager, Research Laboratory, Portland Cement Assn., Chicago, Ill.

[2] Vitruvius, Ten Books on Architecture, Translated by Morris H. Morgan, p. 44, published by Harvard University Press (1914).

> In walls of masonry the first question must be with regard to the sand in order that it may be fit to mix into mortar and have no dirt in it. The kinds of pit sand are these: black, gray, red, and carbuncular. Of these the best will be found to be that which crackles when rubbed in the hand, while that which has much dirt in it will not be sharp enough. Again, throw some sand upon a white garment and then shake it out; if the garment is not soiled and no dirt adheres to it, the sand is suitable.
>
> But if there are no sand pits from which it can be dug, then we must sift it out from river beds or from gravel or even from the sea beach.
>
> After slaking it mix your mortar, if using pit sand, in the proportions of three parts of sand to one of lime; if using river or sea sand, mix two parts of sand with one of lime. These will be the right proportions for the composition of the mixture. Further, in using river or sea sand, the addition of a third part composed of burnt brick, pounded up and sifted, will make your mortar of a better composition to use.

That these recommendations of Vitruvius were followed by the Romans is borne out by data appearing in a recent article by K. Biehl[1] in which compositions and chemical analyses of some ancient German and Roman concretes are described, and their structure illustrated by means of photomicrographs. This article points out that some of the Roman mortars have a large proportion of clean, sharp quartz sand with little broken brick, others the reverse. The lime was completely carbonated in all samples. The mortar with broken-brick aggregate was greatly superior in strength to sand-aggregate mortar. Of German mortars, examined for comparison, one had a quartz sand aggregate, probably a river sand, and the lime had carbonated to the extent of 80 per cent. A second mortar was of similar composition and almost equally carbonated. Both mortars showed small pockets of unmixed lime.

Bry Higgins in 1780 discussed briefly his size classification of sands as follows:[2]

> I cleansed a large quantity of the Thames sand by washing it in streaming water, and sorted it into three parcels; the coarsest, which I call the rubble, consisted of small pebbles, fragments of weathered shells, and grains of sand of divers sizes, which in washing had passed through a sieve whose apertures were $\frac{1}{8}$ in. square, but could not pass through a brass-wired sieve whose meshes were $\frac{1}{16}$ in. square, or rather larger; the next parcel, which I called fine sand, consisted of grains of divers sizes, which in washing passed through a finer sieve whose meshes were $\frac{1}{32}$ in. square; the third parcel consisted of grains the largest of which were washed through the coarsest sieve, and the smallest were retained, in washing, on the fine sieve; this I call coarse sand.

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[1] K. Biehl, "Contribution to the Knowledge of Ancient Mortars," *Tonindustrie Zeitung*, Vol. 52, p. 346 (1928). (See Building Science Abstracts, No. 771, May, 1928.)

[2] Bry Higgins, "Experiments and Observations Made with a View of Improving the Art of Composing and Applying Calcareous Cements," p. 78, published by T. Cadell, London (1780).

It is to be observed that the sand which can pass through a sieve in washing is considerably finer than that which may be sifted through the same sieve when it is dry.

The following paragraphs published in 1837 give Vicat's ideas concerning the desirable properties of sand and their size classification:[1]

Sands being, as we have already said, merely inert substances, it would seem that they ought not to differ in quality from one another in any way except by the form, the size, and the hardness of their grains. The ancient builders wished us to choose the fossil sands, harsh to the touch, in preference to the rounded and polished sands; they regarded the color also, rejected the yellow, etc., etc. But the whole of their writings on that subject are so vague, the experiments on which they depended are so incomplete, and conducted with so little method, that we can conclude absolutely nothing from them. One thing which we know to be quite certain, and which we ought never to lose sight of is this—that there is no sand whatever, be it red, yellow, grey, or white, with round grains or angular ones, etc., which can, if it be inert, form a good mortar with rich lime.[a] Whilst on the other hand, all possible kinds of sand, provided they be pure, and their grains are hard, and do not exceed a certain size, give excellent mortars with the hydraulic and eminently hydraulic limes. Nevertheless, we admit that there are differences in the quality of sands, according as their constituent elements may be granitic, calcareous, schistose, or volcanic; but these differences being in general very small, we shall in what follows merely attend to those which depend upon the size of the grains.

We shall call coarse sand that whose grains, supposing them round, vary from 1½ to 3 mm. (0.059 to 0.0118 in.) in diameter; fine sand, that of which the dimensions in the same way are comprised between 1 and 1½ mm. (0.039 and 0.059 in.); and powders, the solid substances of the same nature, whose largest particles never reach 0.2 mm. (0.00787 in.).

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[a] According to Vicat "rich limes are such as may have their volume doubled, or more, by slaking in the ordinary manner, and whose consistency after many years of immersion remains still the same, or nearly the same as on the first day, and which dissolve to the last grain in pure water frequently changed."

Colonel C. W. Pasley made the following interesting statements in 1838 concerning sand and its measurement:[2]

Though sand deteriorates cement in every respect, yet a moderate proportion of it is always used for the sake of economy, except under critical circumstances in very important works.

In respect to the measurement of sand for the mortar or brickwork or of regular masonry in which the thickness of joints is usually limited, never exceeding ½ in. . . . the coarser parts, such as come under the denomination of "small gravel," being necessarily rejected, by sifting it through a screen, usually having openings not exceeding ⅙ in. in the clear. The chief difference will arise from

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[1] L. J. Vicat, "Practical and Scientific Treatise on Calcareous Mortars and Cements," pp. 85 and 86; translated by J. T. Smith; published by John Weale, London (1837).

[2] C. W. Pasley, "Observations on Limes, Calcareous Cements, Mortars, Stuccos, and Concrete," pp. 203, 216, and 224; published by John Weale, London (1838).

> the state of the sand itself at the period of measurements, which according to our former trials occupied when wet only about four-fifths of the space that it did in its usual rather moist state. On repeating these measurements recently, modified in such a manner as to obtain more precise results, we found that the same quantity of sharp, clean river sand, which in its moist state stood 12 in. high in a regular cubic foot measure, and therefore completely filled it, only stood 10⅜ in. high in the same measure when perfectly dried; and very little more than 8¼ in. high when thoroughly wetted, not by merely pouring water over it in the measure as we had done before, but by mixing every part of the sand with water. Hence, instead of occupying only four-fifths or eight-tenths of its original space, the wet sand now occupied very little more than seven-tenths of the space, which it had previously filled in its moist state.
>
> In respect to sand, the custom is to serve it in by stricken measure, in whatever state it may be at the period of sale, which may vary between more or less dry or wet, according to circumstances, known to the persons who use it, but not to others unless explained, amongst which circumstances the state of the weather has its influence, for sand is not kept under cover, but laid out in masses in the open air. Hence, in order that we may have any precise knowledge of the real proportion which the sand bears to the lime, in the mortar of any work of importance, the person who describes such mortar ought to specify particularly the state in which the sand was measured whether absolutely dry, or damp, or wet; because the actual quantity of sand obtained by the same measure in these three states varies considerably between the second, which is the minimum, and the latter, which is the maximum of quantity. But the sand used for building in this country is scarcely ever in either of these two extremes of perfectly dry or wet. It generally varies only between more or less damp, and probably the difference in real quantity between equal measures of it in these two states, does not exceed one-tenth, in the practice of building at any one place. To describe accurately the state in which it has generally been used for the mortar of any important work, the author should specify not only the mean space occupied by it in that state, but also the spaces which the same quantity of the same sand is capable of occupying when perfectly dry, and when thoroughly wet, stating also its weight per cubic foot when perfectly dry, there being no certainty of the weight of sand in any other state. Moreover, to enable a person who does not know the sort of sand obtained from a particular locality to understand the nature of it, the size of the particles should be described in that way that has been done by M. Vicat, by stating the diameter of the smallest and of the largest grains composing it, the latter of which may be sufficiently defined by describing the sort of screen through which even very fine sand is almost always sifted, to exclude pieces of wood or other extraneous substances generally found in it. When sand and gravel are to be mixed together in any given proportion, the size of both should also be described in the same manner; and even in using some natural mixture of these ingredients, such as Thames ballast, in the mortar of any important work, it is desirable that it should be defined in the same manner. . . .

Q. A. Gillmore in his treatise published in 1871 emphasized the following points:[1]

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[1] Q. A. Gillmore," Practical Treatise on Coignet-Béton and Other Artificial Stones," p. 8, published by D. Van Nostrand, New York (1871).

The sand (for Béton Aggloméré) should be as clean as that ordinarily required for mortar, for stone or brick masonry of good quality. Sand containing 5 or 6 per cent of clay may be used without washing, for common work, by proportionally increasing the amount of matrix. Either fine or coarse sand will answer, or preferably a mixture of both, containing gravel as large as a small pea, and even a small proportion of pebbles as large as a hazel nut. There is an advantage in mixing several sizes together, in such proportions as shall reduce the volume of voids to a minimum. Coarse sand makes a harder and stronger béton than fine sand. The extremes to be avoided are a too minute subdivision and weakening of the matrix, by the use of fine sand only, on the one hand, and an undue enlargement of the volume of voids, by the exclusive use of coarse sand, on the other.

The siliceous sands are considered the best, though all kinds are employed. When special results are desired in the way of strength, texture, or color, the sand should be selected accordingly.

It will be observed that most of the earlier writers on the subject dealt with such questions as suitability of different sands for mortar and concrete and character of particles and grading.

Very little coordinated research work was carried out to determine the influence of the characteristics of fine aggregate on the properties of mortar and concrete until about 1890, when R. Feret began his extensive experiments at the Laboratorie des Ponts et Chaussées at Boulogne sur Mer. Since that time a wealth of data has been published on this general subject and many able investigators have contributed valuable information, among whom may be mentioned: Wm. B. Fuller, S. E. Thompson, D. A. Abrams, L. N. Edwards, R. B. Young, R. W. Crum, A. N. Talbot, F. E. Richart, M. O. Withey, and Otto Graf. The U. S. Bureau of Standards, the U. S. Bureau of Public Roads, and many state highway laboratories have likewise made important contributions to our knowledge of the subject.

This paper covers a general discussion of the function of fine aggregate in concrete, including the significance and tests of the various properties.

FUNCTION OF THE FINE AGGREGATE IN CONCRETE

Aggregates are used in concrete chiefly for reasons of economy, although their presence acts also to prevent undue volume changes which might adversely affect the durability of hardened concrete. Aggregates act as inert fillers increasing the bulk of the mixture and thereby reducing the amount of cement per unit of volume, which effects a corresponding reduction in cost of the product. The function of coarse aggregate is primarily that of economy while the most important functions of fine aggregate are to produce workability and to promote uniformity in the mixture. Fine aggregate is now almost universally considered as that portion of the aggregate below about

$\frac{1}{5}$ or $\frac{1}{4}$ in. in size. The fine aggregate assists the cement-water paste, the binding medium, to float the coarse aggregate particles, thus giving workability to the mass and preventing segregation of coarse aggregate from the paste. Planes of weakness which otherwise would develop along the surfaces of the coarse aggregate particles and influence the strength of the concrete are to a great extent eliminated by the use of a proper amount of fine aggregate. Since workability of concrete is determined largely by the fine aggregate, not only the amount used but also the gradation, size and shape of particles, surface characteristics and other properties affect the placeability of the mix.

CLASSIFICATION OF FINE AGGREGATES

The various kinds of materials now used as fine aggregate in concrete may be classified as (1) natural sands, (2) screenings prepared by crushing natural stones or air-cooled blast-furnace slag ("slag sand"), (3) combinations of screenings and natural sand, (4) lightweight aggregates such as burned shale or cinders, and (5) miscellaneous materials such as shells, crushed fire brick, and granulated blast-furnace slag.

Natural sands are used more extensively than any of the other fine aggregates and, since these have been formed by weathering of rock masses, may consist of any type of rock material. As weathering proceeds the less durable minerals disappear by disintegration and transportation or by solution while the durable minerals are sorted according to their size and specific gravity. Sands that have been subjected to severe weathering action show a marked increase in quartz grains or flint particles, depending upon their source. The ultimate sand, provided weathering acts long enough, consists of the more durable minerals and these are for the most part quartz grains or flint, or mixtures of these together with small percentages of other minerals.

Most sands for concrete are obtained from beds of rivers, from glacial deposits, or from lakes and beaches. Sands containing an excess of clay in lumps or distributed throughout are termed argillaceous. When calcium carbonate is present in considerable quantities the sand is said to be calcareous. Those made up largely of quartz or silicates are siliceous. Sands may frequently present a composition that is a mixture of these various classes. Other more definite terms are frequently used. When mica is present in amounts that are readily discernible, the sand is micaceous. Similarly there are granite sands, garnet sands, chert sands, feldspathic sands, etc.

On projects where suitable natural sands are not readily obtainable, fine aggregates prepared by crushing limestone, granite, trap rock, or sandstone are frequently used. These have given good results where care is taken in selecting the stone and securing a proper gradation of clean particles.

Light-weight fine aggregates are used quite extensively for concrete masonry units and in parts of structures where it is desired to reduce weight. Other materials than those mentioned above have been used as fine aggregate in concrete where special properties are desired or where the concrete was not subjected to severe exposure.

Since the characteristics of fine aggregate have such an important influence on the durability, workability, strength, water-tightness and other qualities of concrete, a large amount of research has been devoted to studies of properties of fine aggregates, to development of methods of test, and to the determination of their influence on the properties of the finished concrete. The three principal considerations involved in the selection of fine aggregate for concrete are (1) composition and structural characteristics of the particles, (2) cleanness of particles and freedom from injurious amounts of deleterious substances, and (3) size and grading of particles.

NATURAL SANDS

Sand has been defined as the fine granular material (usually less than $\frac{1}{4}$ in. in diameter) resulting from the natural disintegration of rock or from the crushing of friable sandstone rocks.[1] In determining the suitability of natural sands for use in concrete, there are a number of factors to be considered besides grading, the more important of which are the following:

Composition:

The mineral composition of a sand is of importance because it determines the structural quality and therefore the durability of sand particles. Since most sands are obtained from river beds, beaches, or glacial deposits, they may include in their composition the entire range of common minerals and rock fragments. Plant remains and carbonaceous material are frequently present. In general, natural sands consist of rounded particles and since one of the functions of sand is to give workability to the concrete mixture the rounded particles are of an advantage, but otherwise the shape of particle has little effect on concrete strength.

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[1] A.S.T.M. Standard Definition of the Term Sand (C 58 - 28), 1928 Supplement to Book of A.S.T.M. Standards, p. 95.

Soundness:

Durability of particles is particularly important in determining the suitability of a sand for use in concrete subjected to severe conditions of exposure. Particles must be of such quality that they will withstand wetting and drying, freezing and thawing, attack by sulfate soils and waters, sea water, and other destructive agencies, both from the point of view of resistance to chemical attack and the disrupting influence of mechanical action produced by the formation of salt crystals in the pores. Siliceous and calcareous aggregates have been found to withstand severe weathering over prolonged periods.

Recent field investigations have shown that in some localities in the northern part of the United States concrete pavements and other exposed structures have shown spalling caused by an expansion of the aggregate particles which in turn has disrupted the concrete adjacent to them. Where these particles were near the surface a pitted, unattractive appearance has been produced. The most unsightly pits were caused by unsound coarse aggregate particles, but pitting was also traced to the coarser particles of fine aggregate. It has been observed in freezing-and-thawing tests of concrete that chert particles sometimes produce this condition.

It has also been observed that when deterioration occurs in hydraulic structures, it takes place at the water line and seems to be closely connected with intermittent wetting accompanied by freezing and thawing. Shale particles expand upon wetting and this expansion is detrimental to concrete wherever the cement paste is of too poor quality to withstand the internal pressures developed.

No very satisfactory method of testing soundness of fine aggregates has yet been developed. The Research Laboratory of the Portland Cement Association has made some experimental studies on sands subjected to alternate soaking in sodium sulfate solution and drying in an oven. These tests were made on particles caught between the following sieves, material finer than the No. 28 sieve being discarded: No. 28 to No. 14, No. 14 to No. 8, No. 8 to No. 4. After 10 or 15 cycles the samples were dried and a sieve analysis made on each. The extent of breaking down of the various sizes of particles was judged by the amount of material passing the sieve on which it was originally held. This procedure was not very satisfactory as it was observed that many particles which showed distress were not sufficiently broken up to pass the sieve and it was necessary to segregate the unsound particles manually. Although the sodium sulfate test is of considerable assistance in judging durability of aggregates, little information is available which permits results to be correlated with effects of freezing-and-thawing tests or behavior of the aggregate in concrete.

Cleanness:

Another important factor to be considered when determining suitability of sand for concrete is cleanness—that is, freedom from coatings and injurious amounts of deleterious substances such as organic impurities, loam, clay, silt, shale, coal, lignite, mica, and soft or rotten particles.

Loam is particularly objectionable in concrete sands as it almost always carries organic materials, sometimes in sufficient quantities to reduce the strength of concrete and destroy the setting properties of the cement.[1] The colorimetric test[2] is now quite generally used in detecting organic impurities in sands. Sands showing a high color in this test should be examined for lignite, as lignite imparts a strong color to sodium hydroxide but does not seriously impair concrete strength. If lignite particles are found in sand a mortar strength test should be made to insure the absence of humous matter in harmful quantities. In the case of a dredged or washed sand, presence of humus is unlikely. Because of the tendency of the light lignite particles to collect near the surface of the concrete, they are likely to accumulate at locations where weathering can affect them, and through them, the concrete. Excessive quantities of lignite may, therefore, impair the durability of concrete even though the strength is not appreciably affected.

Most specifications limit the amount of silt or clay in sands to be used for concrete to 3 per cent by weight. This is a conservative limit but such a low limit has doubtless been set because sands of high silt content are likely to contain organic impurities and even when free from such impurities must be used with caution if laitance seams and scaling are to be avoided and durable surfaces resistant to wear and weather obtained.

Shale particles are particularly detrimental in concrete pavements and in concrete exposed to water and to the weather, because of their poor resistance to weathering and abrasion. F. C. Lang,[3] of the University of Minnesota, has developed a method of determining the amount of shale in coarse aggregate which employs flotation in a zinc chloride solution to separate the shale, and it is possible that this method may be used on the coarser sizes of fine aggregate.

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[1] D. A. Abrams, "Effect of Tannic Acid on Strength of Concrete," *Bulletin* 7, Structural Materials Research Laboratory, Lewis Institute, Chicago.

[2] A.S.T.M. Standard Method of Test for Organic Impurities in Sands for Concrete (C 40 - 27), 1927 Book of A.S.T.M. Standards, Part II, p. 123.

[3] F. C. Lang, "Effect of Shale Pebbles in Concrete and Removal of Shale from Gravel," ***Minnesota Techno-Log***, Vol. 2, January, 1922, p. 11.

Coal particles in sand are objectionable because of their friable nature and poor weathering qualities. The A.S.T.M. Tentative Specifications for Concrete Aggregates (C 33 - 28 T)[1] limit the amount of coal in concrete sand to 1 per cent by weight.

The effect of mica in a concrete sand is dependent upon the form in which it occurs as well as the amount. Unpublished tests made at the Structural Materials Research Laboratory by D. A. Abrams showed that flake mica, as it occurs in natural sands, is more harmful to mortar and concrete strength than a similar weight of pulverized mica. Mortar strength was reduced to a greater extent than concrete strength by the presence of flake mica in the sand. Mica in quantities lower than 1 per cent by weight of sand was not particularly harmful to concrete strength; 1 per cent reduced concrete strength about 15 per cent. To the eye 1 per cent of mica seems a very large amount and most sands which would be considered micaceous contain only a small fraction of 1 per cent. Otto Graf[2] has found that mortars containing 6 per cent of flake mica by weight of cement ($1\frac{1}{2}$ per cent by weight of sand) gave only 60 per cent of the strength of mortar made with sand free from mica.

It is difficult to determine the amounts of coal, lignite, or mica in sands and standard methods of tests have not been developed. A proposed test for coal and lignite by the flotation method using carbon tetrachloride as an agent is given in the 1928 Report of the Society's Committee C-9 on Concrete and Concrete Aggregates.[3] In the tests by Abrams referred to above, the percentage of mica was determined by visual comparison of various micaceous sands with sands of the same fineness modulus containing known percentages of white flake mica.

Sands for concrete should be reasonably free from soft particles as these invariably break up and disintegrate upon exposure to freezing and thawing.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 822 (1928); also 1928 Book of A.S.T.M. Tentative Standards, p. 238.

[2] Otto Graf, "Compressive and Transverse Strength, Shrinkage and Expansion, Wear, Permeability, and Resistance to Chemical Attack of Cement Mortar and Concrete Particularly for Different Gradations and Water Contents of the Mortars," Reprint from *Zement*, Vol. 17, March 15, 22, and 29, 1928, pp. 432, 492, and 543, of paper read at meeting of German Concrete Institute.

[3] Appendix V to the Report of Committee C-9 on Concrete and Concrete Aggregates, *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 365 (1928).

STONE AND SLAG SCREENINGS

The suitability of crushed stone or air-cooled slag screenings ("slag sand") as fine aggregate for concrete has often been questioned because, in a number of instances, concrete made with screenings has proved to be unsatisfactory. Screenings prepared from sound rock or slag and well graded from fine to coarse and free from excessive amounts of dust should give satisfactory results in concrete which is properly proportioned, mixed, placed and cured.

TABLE I.—MISCELLANEOUS TESTS OF SCREENINGS.

Lot	Source	Sieve Analysis, Retained on Each Sieve, per cent by weight							Fineness Modulus	Surface Area, sq. in. per lb.	Unit Weight, lb. per cu. ft.	Specific Gravity	Absorption, per cent by weight	Silt, per cent by weight
		No. 100	No. 48	No. 28	No. 14	No. 8	No. 4	$\frac{3}{8}$ in.						
LIMESTONE SCREENINGS														
No. 3435	Madison, Wis.	77	64	52	43	26	9	0	2.71	5150	114	2.53	2.96	15.8
No. 3452	Racine, Wis.	92	91	87	80	60	24	0	4.34	1780	100	2.70	1.20	11.6
No. 3457	Menard, Ill.	78	70	63	51	30	9	0	3.01	4640	105	2.48	3.26	21.2
No. 3573	Chicago, Ill.	86	80	73	62	40	14	0	3.55	3110	114	2.66	1.96	19.3
No. 3812	Howes Cove, N. Y.	78	71	58	42	15	2	0	2.66	4620	122	2.62	1.19	20.7
No. 3815	Greenleaf, Wis.	83	80	75	68	50	18	0	3.74	3150	91	2.69	1.86	10.3
No. 3819	Milwaukee, Wis.	79	67	54	41	20	4	0	2.65	4530	115	2.74	0.91	16.7
No. 3831	Omaha, Neb.	98	90	83	66	28	5	0	3.70	1850	117	2.59	0.42	0.1
No. 3957	Chicago, Ill.	87	80	68	43	24	2	0	3.04	2950	110	2.68	1.68	19.0
No. 4492 A	Chicago, Ill.	84	77	65	50	25	8	0	3.09	3670	114	2.66	1.72	12.9
No. 4984	Montrose, N. Y.	63	54	39	32	18	2	0	2.08	6590	121	2.66	2.04	18.0
TRAP SCREENINGS														
No. 3438	Dresser Jct., Wis.	88	81	67	43	1	0	0	2.80	3360	118	2.71	2.47	15.5
No. 3467	Howellsville, Pa.	80	73	67	57	39	19	0	3.35	4000	123	2.59	2.24	20.7
No. 3486	Marquette, Mich.	91	87	84	77	54	4	0	3.97	2080	110	2.76	1.44	15.0
GRANITE SCREENINGS														
No. 3821	Red Granite, Wis.	78	69	56	38	9	0	0	2.50	4940	113	2.54	1.52	15.6
No. 3840	Berlin, Wis.	82	72	67	49	20	5	0	2.95	4310	109	2.57	1.42	11.7
No. 4081	Red Granite, Wis.	84	80	72	62	36	4	0	3.38	3400	106	2.50	2.08	9.7
SLAG SCREENINGS														
No. 4114	Youngstown, Ohio	94	86	75	61	43	21	0	3.80	2410	92	2.53	2.51	5.7
No. 4137	South Chicago, Ill.	93	84	66	51	28	2	0	3.24	2880	92	2.57	2.52	2.1
No. 4239 A	South Bethlehem, Pa.	94	89	78	57	29	5	0	3.52	2510	89	2.41	3.92	4.7
No. 4554	Buffington, Ind.	91	83	61	25	2	1	0	2.63	3840	82	2.34	4.66	0.3
No. 4555	Buffington, Ind.	83	72	50	11	0	0	0	2.16	4960	90	2.48	3.31	10.6
No. 4938	North Chicago, Ill.	100	99	97	92	54	4	0	4.46	840	116	2.46	11.16	0
No. 5051	Chicago, Ill.	95	92	84	76	57	29	0	4.33	1760	94	2.41	2.48	....

In screenings, the particles have rough surfaces and are of angular, elongated or splinter shape, depending on the type of rock from which they are crushed, and therefore do not produce as workable a mix as most natural sands when used under otherwise similar conditions. Because of the roughness and angular shape of the particles and the frequently poor grading of the usual run of screenings, they require more mixing water than natural sands to produce the desired consistency. Therefore they necessitate the use of more cement when the concrete is designed on the basis of a fixed water-cement ratio and consistency.

Table I gives the results of sieve analyses and other tests on a number of samples of screenings from different types of material. The sieve analyses in this table show that screenings are frequently coarsely graded and contain large amounts of material passing the No. 100 sieve. These gradings can be corrected by washing to reduce the amount of dust and by adding the sizes which are lacking.

Excessive amounts of dust are to be avoided as this dust is largely responsible for lack of durability and wearing resistance in concrete. D. A. Abrams[1] reports that "dust in screenings in quantities up to, say, 10 per cent does no harm so long as the concrete is well mixed and of a proper consistency; too much mixing water causes the dust and other fine material in the concrete to rise to the surface where it forms a weak friable layer called laitance, or produces a concrete which is unable to resist wear or weather." The conditions mentioned in the last part of Abrams' statement have been found to cause scaling of concrete roads and other finished surfaces and hence are to be avoided.

A mixture of natural sand and stone screenings will often produce a very satisfactory fine aggregate, particularly if the sand is so selected as to supply any deficiencies which may exist in the grading of the screenings.

In the preparation of stone screenings for use in concrete, care should be taken to eliminate quarry dirt.

The suitability of screenings for use in concrete is frequently determined from a comparison of their tensile or compressive strength in 1:3 mortar with that of standard Ottawa sand mortar of the same consistency. Screenings, even when they contain a large amount of dust, usually show high mortar strength as compared with Ottawa sand, due primarily to their generally coarse grading which tends to reduce the quantity of mixing water required for the given consistency. Also because of the angularity of the particles there is a tendency for the operator to compact the mortar more when molding the specimens. In mortar tests screenings may show from 10 to 90 per cent higher strengths than standard Ottawa sand and yet not be satisfactorily graded for best results in concrete.[1, 2] D. A. Abrams reports that the strength of concrete made from screenings on the same basis is somewhat lower than that from natural sands.[1]

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[1] D. A. Abrams, "Stone Screenings as Fine Aggregate for Concrete," *Rock Products*, Vol. XXV, No. 5, March 10, 1923, p. 131.

INFLUENCE OF SIZE AND GRADING OF FINE AGGREGATE ON PROPERTIES OF CONCRETE

Strength:

An important element in determining the concrete-making properties of fine aggregates is the gradation of particles, and many studies have been carried on both in this country and in Europe to determine the influence of size and grading of fine aggregate on the properties of mortar and concrete. R. Feret was the first to carry out any extensive investigation to discover the relationships between strength and volumetric composition of mortars. His law for mortar strength published in 1897[3] is the earliest statement of the cement-void ratio - strength relation. Feret's researches on mortars were extended to concrete by a number of other investigators, notably, Wm. B. Fuller and S. E. Thompson, A. N. Talbot and F. E. Richart, and R. W. Crum. Feret found that variations in size and grading of sand particles affected the quantity of mixing water required to produce the same plasticity in a given mix and, therefore, influenced the voids (air and water) and strength of the resulting mortar. The most advantageous grading was obtained when the sand contained approximately 2 parts coarse material to 1 part fine material including the cement, with the least possible amount of medium-sized grains. Except for "puzzolans" Feret found little difference in strengths of mortars made with siliceous, calcareous, or dolomitic sands.

Talbot and Richart[4] in extending the work of Feret to the design of concrete mixtures pointed out the influence of the physical make-up of the mortar on concrete strength and the relationships which exist between concrete strength and voids in concrete. R. W. Crum applied Feret's principles to the design of concrete mixtures in which gravel aggregates containing a large proportion of sand were used.[5] An outgrowth of Fuller and Thompson's studies of aggregate gradings was the development of what Fuller called the ideal gradation curve for the best mixture of fine and coarse aggregate of given maximum size.[1]

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[1] D. A. Abrams, "Stone Screenings as Fine Aggregate for Concrete," *Rock Products*, Vol. XXV, No. 5, March 10, 1923, p. 131.

[2] P. J. Freeman, "Selecting Fine Aggregates," *Concrete*, Vol. 20, February, 1922, p. 79.

[3] *Bulletin*, Société d'Encouragement pour l'Industrie Nationale, Vol. II, p. 1604 (1897).

[4] A. N. Talbot and F. E. Richart, "The Strength of Concrete: Its Relation to the Cement Aggregates and Water," *Bulletin No. 137*, Engineering Experiment Station, University of Illinois (1923).

[5] R. W. Crum, "Proportioning of Pit-run Gravel for Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. XIX, Part II, p. 458 (1919).

Otto Graf, a well-known German investigator, has also studied the effect of granulometric composition of mortars on the properties of concrete. In Graf's opinion strength of concrete is determined primarily by the strength of the mortar. Therefore he has endeavored to find that grading of the dry mortar (sand and cement) which gives highest mortar strength. In a recent publication he gives the following composition of the dry mortar as the optimum combination of sizes for any concrete mix:[2]

25 per cent passing 0.24-mm. (0.0093-in.) sieve opening
35 " " " 1.0-mm. (0.0394-in) diameter screen opening
65 " " " 3.0-mm. (0.118-in.) " " "
100 " " " 7.0-mm. (0.276-in.) " " "

Since this grading includes the amount of cement in the sieve analysis, it follows that a lean mix should contain more of the finer sand particles than a rich mix where a large part of the fines is supplied by the cement. In later tests Graf reports that highest compressive and flexural strength as well as the least volume change and permeability is obtained when the granulometric composition of the mortar corresponds to his best gradation curve.[3] He recommends that this gradation curve be used only as a general guide in selecting fine aggregates for the individual job.

The findings of these various investigators are in close agreement with Abrams' well-known water-cement ratio law for concrete mixes which holds that for plastic mixes and sound aggregates, the strength and other properties are governed by the water-cement ratio.[4] According to this law, size and gradation of fine aggregate affect strength, wear, and other properties only as they affect the quantity of water required for a given consistency. The coarser the fine aggregate the higher is the strength of concrete of a given mix and consistency due to the fact that less mixing water is required for coarse, well-graded sand than for fine or poorly graded sand. Fine sand, therefore, decreases

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[1] W. B. Fuller and S. E. Thompson, "The Laws of Proportioning Concrete," *Transactions*, Am. Soc. Civil Engrs., Vol. LIX, p. 67 (1907).

[2] Otto Graf, "Der Aufbau des Mortels und des Betons," 2nd Ed., published by J. Springer, Berlin (1927).

[3] Otto Graf, "Compressive and Transverse Strength, Shrinkage and Expansion, Wear, Permeability, and Resistance to Chemical Attack of Cement Mortar and Concrete Particularly for Different Gradations and Water Contents of the Mortars," Reprint from *Zement*, Vol. 17, March 15, 22, and 29, 1928, pp. 432, 492, and 543, of paper read at meeting of German Concrete Institute.

[4] D. A. Abrams, "Design of Concrete Mixtures," *Bulletin 1*, Structural Materials Research Laboratory, Lewis Institute, Chicago.

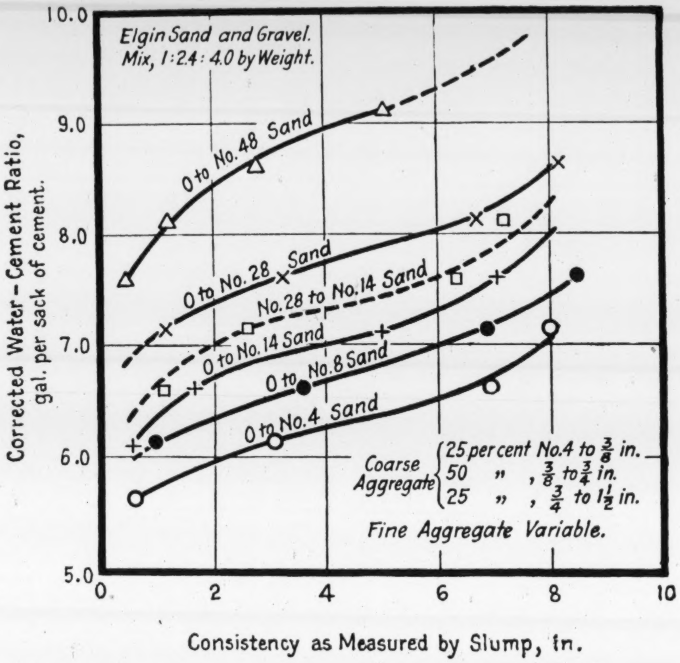

FIG. 1.—Effect of Size and Grading of Fine Aggregate on Slump of Concrete of Constant Mix.

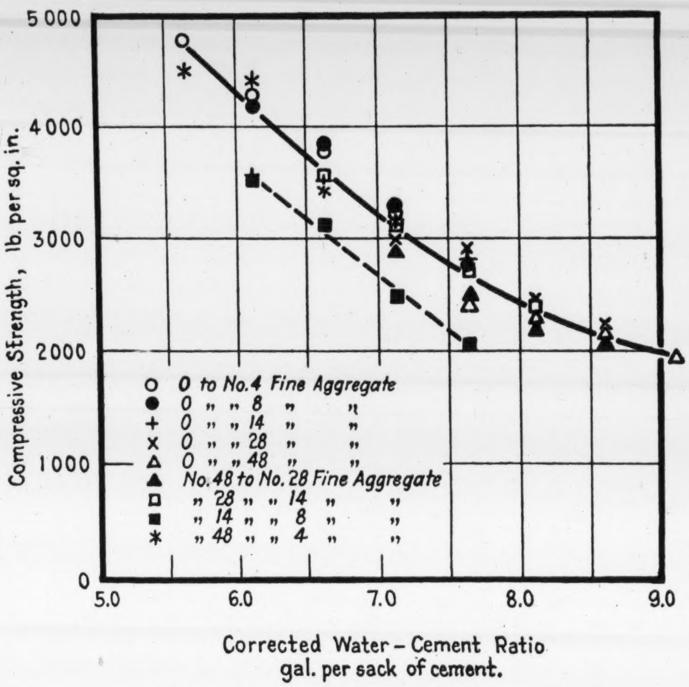

FIG. 2.—Water-Cement Ratio - Strength Relations of Concrete with Different Sizes and Gradings of Fine Aggregate.

concrete strength on account of the higher water-cement ratio required. The effect of fine sand is partly offset by the fact that the amount of sand can be somewhat reduced without producing a harsh mix. However, since decreasing the amount of fine sand does not entirely offset the increased water requirement, the final result will be a reduction in strength.[1]

Another factor which reduces the strength of concrete made with very fine sand is too great a gap in grading between the fine and coarse aggregate. Tests have shown that the larger this gap the lower will be the strength for given proportions. Therefore, the best grading of mixed aggregate is that in which all sizes are represented ranging from the very fine sand particles to the maximum size particles of coarse aggregate.

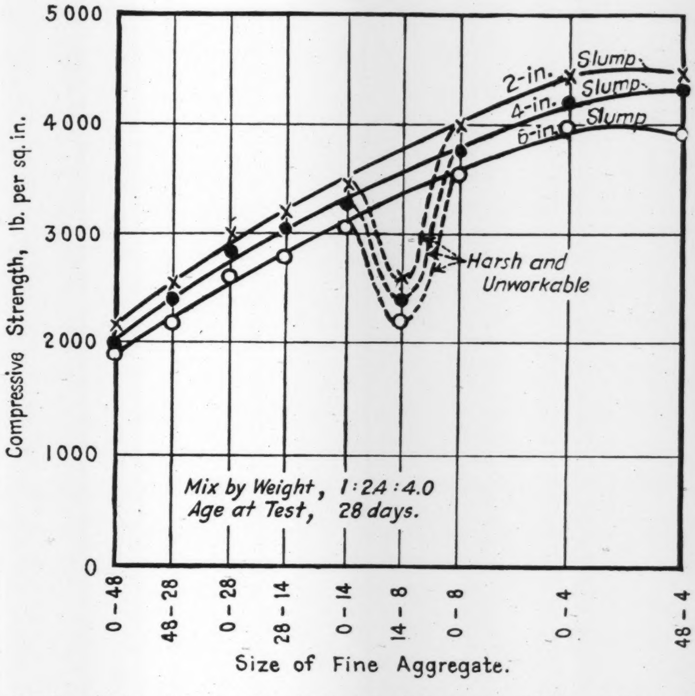

FIG. 3.—Effect of Fine Aggregate on Strength of Concrete of Given Mix and Slump.

Results of recent tests made at the Research Laboratory of the Portland Cement Association, Chicago, are shown in Figs. 1, 2, and 3. In these tests fixed proportions by weight were used, the size and grading of fine aggregate being varied over a large range while the size and grading of coarse aggregate remained constant. Fig. 1 shows the marked effect of size and grading upon the water required for a given consistency as measured by the slump. Increasing the fineness of the sand by screening out the coarser sizes required a considerably higher water-cement ratio to maintain a given slump. Changing the sand from a relatively coarse (0 to No. 4) to a very fine (0 to No. 48) required an increase in the water-cement ratio of 2 to $2\frac{1}{2}$ gal. per sack in order to give the same slump.

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[1] Report of Director of Research, Portland Cement Association, November, 1928, p. 27.

Data from the same series are plotted in Fig. 2 which shows that the water-cement ratio - strength relation was not appreciably affected by changes in size and grading so long as the mix remained plastic and workable. All gradings except the No. 14 to No. 8 sand, which gave a very harsh, unworkable mix, follow closely the same water-cement ratio curve. In Fig. 3 the same data from Fig. 2 are plotted to show the relation between concrete strength and size of fine aggregate for slumps of 2, 4, and 6 in. A smooth curve is obtained for each slump, the only points not following the curves are those for the very harsh mixes containing No. 14 to No. 8 sand. Figure 3 shows that for fixed proportions and slump, an improvement in size and grading of the fine aggregate may increase concrete strength as much as 100 per cent. Thus size and grading of fine aggregate are very important factors as regards economy.

Permeability:

Size and gradation of fine aggregate have the same influence upon permeability, durability, and other properties of concrete as on strength. The quality of the mortar and concrete is fixed by the quality of the cement paste, and those gradings of fine aggregate which will be plastic and workable with the driest paste (lowest water-cement ratio) will yield the best mortar or concrete. This has been demonstrated so far as permeability is concerned by the experiments of Otto Graf and Gustav Merkle. Graf found that the gradation of the mortar which gave the maximum strength also produced the most impermeable mortar.[1] He also reports that stone screenings with the same cement content gave a more permeable mortar than did glacial or river sands. Merkle states that:[2]

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[1] Otto Graf, "Compressive and Transverse Strength, Shrinkage and Expansion, Wear, Permeability, and Resistance to Chemical Attack of Cement Mortar and Concrete Particularly for Different Gradations and Water Contents of the Mortars," Reprint from *Zement*, Vol. 17, March 15, 22, and 29, 1928, pp. 432, 492, and 543, of paper read at meeting of German Concrete Institute.

[2] Gustav Merkle, "Wasserdurchlässigkeit von Beton," published by J. Springer, Berlin (1927).

The permeability of concrete, like the compressive strength, is primarily dependent on the water-cement factor; that is, over the range of plastic to fluid consistencies the permeability increases with the increasing water-cement factor. The kind of aggregate, whether of gravel or crushed material, is of no essential concern, assuming that the same water-cement factor is used. The sand content should be so regulated that the coarse aggregate is well surrounded with mortar. Of significance, although in a lesser degree, is a good grading of the sand in which dust particles up to a certain amount should be present. On the other hand, however, the results of these experiments did not confirm the opinion that sand, especially the dust particles, when used in excess are suitable for filling the pores and decreasing the permeability. When used in excess they rather increase, not only the permeability of concrete due to the leaner mix but they also increase the water requirements of the aggregate in order to obtain a definite consistency, and consequently, have an unfavorable influence upon the water-cement factor. In the light of this series, with a maximum size of 60 mm., a sand content of the aggregate of 40 per cent 0 to 8-mm. size, and 15 per cent 0 to 0.5-mm. size proved to be most satisfactory.

In general, that grain composition (grading) which requires the least water can be considered the most suitable with the limitation however that the sand content be not too greatly reduced. On this basis, as regards water-tightness, the gravel concrete is preferable to concrete of crushed material. The same amount of water which is required for obtaining plastic consistency with the sandstone concrete suffices to produce a fluid mixture out of gravel concrete of the same composition of grain.

Workability:

It is difficult to establish rules for grading of fine aggregate without knowing the grading of the coarse aggregate with which it is to be used. Frequently the product of commercial sand-producing plants has a relatively small amount of material between the No. 14 and No. 4 sieves. This will produce a well-graded mixed aggregate when combined with coarse aggregate having some particles smaller than the No. 4 sieve. When sand having a large percentage of coarse particles is used, it is important that the coarse aggregate have little or no material of the size of the maximum sand particles, otherwise a so-called "grainy" concrete, difficult to finish, will result. This may occur when screenings are used.

In order to give smoothness and workability to the concrete, it is also important that the sand contain a sufficient amount of fines. When sand is deficient in fines, that is, material passing the No. 50 or No. 28 sieve, increasing the sand percentage (the common method of securing smoother working concrete) will not produce real workability or prevent segregation except in rich mixes. The minimum percentages passing the No. 50 sieve as recommended or specified by various organizations are as follows:

1924 Report of Joint Committee on Concrete and Reinforced Concrete	10 per cent
U. S. Master Specification No. 464	5 per cent
19 state highway departments	5 per cent or more
A.S.T.M. Specifications C 33 – 28 T	2 per cent

Fines in sand must not be confused with silt, loam or other impurities, which are undesirable as pointed out above.

TABLE II.—TYPICAL REQUIREMENTS IN SPECIFICATIONS FOR FINE AGGREGATE.

Item	1924 Report of Joint Committee on Concrete and Reinforced Concrete	A.S.T.M. Specifications C 33 – 28 T	American Concrete Institute	U. S. Master Specification No. 464	State Highway Departments[a]
GRADING REQUIREMENTS					
Maximum size sieve	No. 4	No. 4	No. 4	¼-in. Screen	(47)
Per cent passing	85 to 100	85 to 100	85 to 100	95 to 100	
Intermediate size sieve	..	No. 16	No. 16	..	(37)
Per cent passing	..	45 to 80	45 to 80	..	
Per cent passing No. 50 sieve, minimum	10	2	2	5	5 or more (19)
Per cent passing No. 50 sieve, maximum	30	30	30	30	(39)
Per cent passing No. 100 sieve, maximum	..	5	5	8	(44)
IMPURITIES					
Silt, maximum per cent by weight	3	3	3	3	(45)
Shale, maximum per cent by weight	..	1	1	..	(2)
Coal, maximum per cent by weight	..	1	1	..	(1)
Clay lumps, maximum per cent by weight	..	1	1	..	..
Sum of 4 previous items, maximum per cent	..	5	5	..	..
Colorimetric test; not darker than standard color	Yes	Yes	Yes	Yes	(28)
STRENGTH TESTS					
Mortar strength compared to standard Ottawa briquets or cylinders of same consistency, per cent	100	100	100	..	(48) 100 (39)
Concrete strength, per cent	..	..	100	..	(12)

[a] The numbers in parentheses in this column indicate the number of State Highway Departments which have some definite requirement in their specifications for the item shown.

In commercial work it is necessary to make the best use of available materials, so that grading should be restricted only to prevent extreme conditions. Under the water-cement ratio method of designing concrete mixtures, these limitations may be more liberal than under the old arbitrary system of proportioning which required fairly well-graded aggregates. In some cases it may even be economical to use poorly graded sand, otherwise suitable, making up for deficiencies in grading by an increased cement content. In some localities the practice is to mix (blend) sands or to use admixtures to improve grading.

SPECIFICATIONS FOR FINE AGGREGATE

Most specifications for fine aggregate contain a general clause to the effect that they shall consist of clean, sound, durable particles of inert materials. In addition, certain restrictions are placed on grading and amount of impurities, and provision is made for a mortar strength test. Table II covers the more common clauses in several widely used specifications.

It will be noted that there is a tendency to liberalize permissible gradings with the exception of the necessary fines to secure workability. A provision is sometimes included that sands lacking in fines may be used if the deficiency is corrected by addition of fine sand from another source.

Liberal tolerances in grading are possible because of the gradual abandonment of the fixed-mix system of proportioning in favor of designed mixes. A clause sometimes incorporated in specifications permits the use of aggregate materials not conforming to the specification requirements provided equivalent concrete strengths can be obtained. The first attempt to correlate a given fine aggregate with its accompanying coarse aggregate is contained in the present A.S.T.M. Tentative Specifications for Concrete Aggregates (C 33 - 28 T)[1] which prohibits "overlapping" of material near the $\frac{1}{4}$-in. size.

The colorimetric and silt tests assure satisfactory quality so far as cleanness is concerned with the strength requirement as a check. A recent tendency is to limit the total amount of impurities of all types. Other impurities are sometimes taken account of in local specifications, such as salt content for reinforced concrete near the coast.

In some cities general specifications have been adopted which take into account local conditions. Such specifications are desirable to prevent confusion of requirements and to reduce the number of stock sizes to a minimum.[2] Obviously special requirements increase the cost of production.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 822 (1928); also 1928 Book of A.S.T.M. Tentative Standards, p. 238.

[2] L. E. Williams, "Confusion of Specifications for Aggregates," *Proceedings*, Am. Concrete Inst., Vol. XXV, p. 642 (1929).

FINE AGGREGATE IN MORTAR AND PLASTER

By J. C. Pearson[1]

Synopsis

This paper presents a brief discussion of the functions and properties of ordinary mortar and plaster, and shows the relatively minor and more or less indeterminate contribution of the fine aggregate to these properties. Hence the existing lack of attention to quality of fine aggregate for mortar and plaster. Existing specifications for lime and gypsum plastering sands are not entirely satisfactory, as they undoubtedly debar many suitable aggregates. Data most needed for a logical selection of aggregate have to do with plasticity and volume change, but very few have been secured up to this time and more fundamental studies are needed to supply such data.

The application of the so-called plastic mortar test to 35 building sands indicates that this is a safeguard against dangerous sands, and that poorly graded or excessively fine sands are uneconomical. However, a complete and satisfactory specification for fine aggregate can hardly be written until general relations are established between the characteristics of fine aggregates and the properties of plasticity and volume change in mortars and plasters.

This discussion will be confined entirely to the properties of fine aggregate suitable for ordinary mortar and plaster. The reader will attune himself most readily to the somewhat liberal views expressed herein if he accepts the premise that ordinary mortars and plasters have certain functions to perform which are not mainly dependent on their strength, and that, in serving these functions properly, their ingredients are not logically subject to the restrictions that apply when strength and durability in the face of severe exposure are the main considerations.

Functions of Mortar and Plaster

Most of the ordinary mortar, and by this we mean masonry mortar, is used for bedding and bonding together masonry units of various kinds—stone, brick, tile, concrete blocks, etc. The stresses that come upon it are mostly vertical, and in the thin, restrained layers in which it occurs, a low order of compressive strength only is sufficient for the imposed loads. Durability is required, of course, but the question of durability hardly arises except in foundations, or locations more or less severely exposed to water. The frequent disintegration of masonry work under these conditions is evidence enough of the special requirements that have to be met in such cases, and they are beyond the scope of this paper. It appears therefore that ordinary mortar will serve its purpose best if it is plastic, if it does not shrink excessively, if it is reasonably free from soluble salts that may produce stain or efflorescence, and if it is capable of developing a fair degree of strength and hardness. These are the chief requirements, and in the author's opinion, they are listed in the order of their importance. Not all may agree with this opinion, and it must be admitted that a number of investigators, for example, Johnson[1] and Weymouth,[2] have seemed to place more emphasis on strength in certain types of masonry mortars than necessary. This may be due, more than anything else, to traditional influences in the art of testing, which have fostered a sort of unreasoning faith in the general relation between strength and quality of materials, regardless of particular uses. On the other hand, Bates has supported the author's opinion in his excellent review of "Modern Cements,"[3] and those who have had experience in testing some of the masonry cements now on the market must realize that high strength is not the reason for the popularity of these products.[4]

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[1] Assistant to Chemical Engineer, Lehigh Portland Cement Co., Allentown, Pa.

Plaster is used mainly for decorative purposes, and its chief function is to "stay put" and look well. It serves its purpose best if it is plastic, if it does not shrink excessively, if it is reasonably free from soluble salts that may produce stain or efflorescence, and if it is capable of developing a fair degree of strength and hardness. In other words, the chief requirements of an ordinary plaster, and by this we mean an interior plaster whose function is mainly decorative, are identical with those of a masonry mortar, and if they differ at all, it is only in the degree of importance, which may vary according to local conditions.

It is unfortunate that these properties of mortar and plaster are very difficult to measure, with the single exception of strength, which, as has been stated, is relatively unimportant. In regard to plasticity, the difficulties begin with trying to define this most important property, and if we accept the definition now tentatively before the Society[1] we may declare that there is no recognized method nor present available means for determining the "complete force-flow relation" in mortars and plasters. We must develop more fundamental methods before we can interpret with certainty the indications of the various plasticimeters, viscosimeters, penetrometers and other devices which have been used heretofore in attempts to measure the plasticity of mortars. In regard to shrinkage, quantitative data can be obtained under certain artificial conditions, but only with difficulty and great uncertainty can we reproduce the actual conditions which exist in a structure. For example, it is well known that the properties of a mortar or plaster are affected by the rapid loss of water through "suction" of the masonry units or backing to which it is applied. Tests by the author some years ago[2] showed that shrinkage of stucco mortars was greatly reduced in this manner; more recently Davis and Troxell[3] have shown a similar behavior of masonry mortars, on which they comment as follows: "It is clear that the behavior of mortar cast in a non-absorbent mold is no criterion to the behavior of the same mortar when employed in brick work."

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[1] H. V. Johnson, "Sands for Lime, Gypsum and Cement Plasters," *Rock Products*, Vol. 28, p. 54 (1925).

[2] L. E. Weymouth, "Improved Brick Mortars," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 446 (1927).

[3] P. H. Bates, "Modern Cements," *Engineering News-Record*, Vol. 100, June 7, 1928, p. 887; and June 14, 1928, p. 932.

[4] At the time this was written, the U. S. Government Master Specification for Masonry Cement, No. 443 (U. S. Bureau of Standards *Circular No. 321*, February, 1927), had not come to the author's attention. The strength requirement in these specifications is that the average compressive strength of three 2-in. cubes made of mortar composed of 1 part masonry cement and 3 parts standard Ottawa sand by weight, when mixed with sufficient water to give a flow of 100 to 115 on the 10-in. flow table, shall be not less than 175 lb. per sq. in. at 7 days; also that the average 28-day strength shall not be less than the 7-day strength.

Finally we come to the most hopeless of all problems—that of setting some kind of a limit on the ability or tendency of a mortar to effloresce. Since every type of cementitious material that is used in ordinary mortar or plaster is in some degree soluble in water, and therefore has in itself a potential capacity for efflorescence, it does not seem within the bounds of probability that any definite, limiting clause in regard to efflorescence is likely to be embodied in a specification for mortar or plaster—certainly not in the near future. The entire subject is much less simple than it might seem, and there is not space for further discussion of it here. For those who are interested in the causes and methods of prevention of efflorescence on limestone and brick, comparatively recent papers by Huber[4] and Palmer[5] contain very pertinent information. If some one could equally well explain the occurrence or non-occurrence of efflorescence on a variety of cement and concrete products under given circumstances, it would be a great benefit to the industry.

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[1] Definition published as information by Technical Committee on Plasticity, Consistency, Etc., in the Report of Committee E-1 on Methods of Testing, *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 621 (1928), as follows:

Plasticity is that property of a material by which, as the shearing stress is lowered, the material resists change of shape relatively more than in the case of a fluid; thus for a plastic material the ratio of flow to force is not constant.

[2] J. C. Pearson, "Shrinkage of Portland Cement Mortars, and Its Importance in Stucco Construction," *Proceedings*, Am. Concrete Inst., Vol. 17, p. 133 (1921).

[3] R. E. Davis and G. E. Troxell, "Volumetric Changes in Mortars and Concretes," *Proceedings*, Am. Concrete Inst., Vol. 25 (1929).

[4] Lee Huber, "Staining and Efflorescence on Indiana Limestone Caused by Moisture Seepage Through Backing Masonry Materials," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 695 (1928).

[5] L. A. Palmer, "Wet Walls and Efflorescence," Publication of the American Face Brick Association containing results of research work at the U. S. Bureau of Standards, issued 1928.

SPECIFICATIONS

To sum up the situation as it has been set forth in the previous discussion, we recognize four main properties of ordinary mortars and plasters for which specification limits might desirably be set, as follows:

1. Plasticity,
2. Volume change,
3. Content of water soluble material, and
4. Strength.

It has been shown that data are not now available for setting up limitations on the three most important of these properties and the fourth is so relatively unimportant that a very low order of strength would be sufficient. Furthermore, it may be assumed that these properties of mortars and plasters are dependent mainly on the properties of ingredients *other* than the fine aggregate. In the face of this situation, how can a specification for fine aggregate for mortar or plaster logically be written? We know only one thing with certainty, namely, that fine aggregate serves as a filler, and as such the usual characteristics of hardness, inertness, etc., are probably desirable. We have no positive information as to the effect of the grading of fine aggregate, except on strength, which varies widely from one type of mortar or plaster to another and depends far more on the type of cementitious material than on the type of aggregate. We have some general ideas on workability as affected by the various ingredients, and subscribe to the statements of Smith and Conahey,[1] Bates and Dwyer[2] and others, in so far as these apply to fine aggregate mixtures, but we have nothing more definite in regard to the effect of the aggregate itself than Goldbeck's statement,[3] which well summarizes the existing knowledge: "In general, the finer the fine aggregate, the greater will be the workability."

In this general lack of definite knowledge it is not surprising, as Walker and Proudley[4] state, that "No specification has been proposed as yet by any of the national engineering organizations for sand for brick mortar." There are, however, two specifications for sand for plaster sponsored by this Society, and we shall examine them briefly in detail. The first of these is entitled Standard Specifications for Gypsum Plastering Sand (C 35 – 25),[1] and the second Tentative Specifications for Sand for Use in Lime Plaster (C 66 – 27 T).[2]

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[1] G. A. Smith and George Conahey, "A Study of Some Methods of Measuring Workability of Concrete," *Proceedings*, Am. Concrete Inst., Vol. 24, p. 24 (1928).

[2] P. H. Bates and J. R. Dwyer, "Cement as a Factor in the Workability of Concrete," *Proceedings*, Am. Concrete Inst., Vol. 24, p. 43 (1928).

[3] A. T. Goldbeck, "Gradation and Character of Aggregates as a Factor in Workability," *Proceedings*, Am. Concrete Inst., Vol. 24, p. 56 (1928).

[4] Stanton Walker and C. E. Proudley, "Representative Specifications for Different Uses of Sand and Gravel," *Bulletin No. 3*, Nat. Sand and Gravel Assn., November, 1928.

A composite of the general requirements of these two specifications is that the sand shall consist of hard, strong, durable, uncoated mineral or rock particles, containing not less than 80 per cent by weight of silica, feldspar, dolomite, magnesite or calcite, and shall be free from saline, alkaline, organic or other deleterious substances. It is of course all right to set down in a specification one's idea of what a fine aggregate ought to be, and while one is about it, he may as well describe an ideal, as nearly as may be; but as a mandatory paragraph of the specification, such an enumeration is ridiculous.

In gypsum plastering sand not more than 6 per cent shall be retained on a No. 8 sieve, not less than 80 per cent on a No. 50 sieve, and not less than 94 per cent on a No. 10 sieve. In regard to this grading, it is the author's belief that most plasterers prefer not to use sand containing any considerable quantity of material coarse enough to be retained on a No. 8 sieve. The 6 per cent in these specifications may therefore be considered a reasonable tolerance. It is a question, however, whether a maximum limit of 20 per cent finer than a No. 50 sieve will in all cases give desirable workability. Pearson and Hitchcock[3] found that the workability of concrete mixtures containing a fine aggregate of which 20 per cent passed the No. 50 sieve was distinctly better than one of which 15 per cent passed the No. 50 sieve (fineness moduli 2.71 and 2.91, respectively). If this change were noticeable in concrete mixtures, it would presumably be even more noticeable in mortar mixtures, and our general experience with the advantages of particular gradings of aggregates in mortars and concretes indicates that the inclusion of a larger fraction below the No. 50 sieve would improve workability and should be permissible. In comemnting upon this limitation the U. S. Bureau of Standards states:[4] "The requirement that 80 per cent of the sand shall be retained on a No. 50 sieve shall be regarded as an ideal to be approached rather than a rule to be enforced." The question which the author would raise is whether this 80-per-cent limitation is an ideal, but in the lack of any suitable means for measuring workability, the question cannot be settled.

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[1] 1927 Book of A.S.T.M. Standards, Part II, p. 82.

[2] *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 774 (1927); also 1928 Book of A.S.T.M. Tentative Standards, p. 209.

[3] J. C. Pearson and F. A. Hitchcock, "Economic Value of Admixtures," *Proceedings*, Am. Concrete Inst., Vol. 20, p. 312 (1924).

[4] "Wall Plaster, Its Ingredients, Preparation and Properties," U. S. Bureau of Standards *Circular No. 151* (1924).

The most constructive work that has been done on the grading of plaster sands was done under the auspices of Committee C-7 on Lime, the results of which were the basis of its Specifications for Sand for Use in Lime Plaster (C 66 – 27 T). This committee was instrumental in collecting data on 64 typical plastering sands from 30 states, all of which were considered acceptable in local use. The gradation limits set by the committee as a result of these studies are as follows:

	UPPER LIMIT	LOWER LIMIT
Percentage Retained on No. 8 Sieve	10	0
Percentage Retained on No. 16 Sieve	45	8 (interpolated)
Percentage Retained on No. 30 Sieve	80	15
Percentage Retained on No. 50 Sieve	95	70
Percentage Retained on No. 100 Sieve	100	95
Percentage Removed by decantation	5	0
Fineness Modulus	3.30	1.88

This places the limits rather wide, and yet about one-third of the 64 sands failed to comply with these limitations in all respects. The finest had a fineness modulus of about 1.20, the coarsest a fineness modulus of about 3.70. While the committee was undoubtedly wise in excluding these very fine and very coarse sands, the question still remains whether some of the sands that failed to comply with the gradation requirements were not actually more suitable than some of those that fell within the prescribed limits. The answer to this question is probably to be found only in studies of shrinkage and plasticity, the one being very difficult and the other practically impossible to carry out in the present state of the art.

TESTS

From all that has been said up to this point, one may infer that the significance of the usual tests of fine aggregate for use in ordinary mortar or plaster is practically nil. In the author's opinion it is doubtful whether there is any justification at the present time for setting gradation limits for plastering sand, even such as the wide boundaries of Specifications C 66 – 27 T. Data presented later on 35 fine aggregates of widely varying gradations support this contention, but we are quite in the dark without definite information on the general relations between characteristics of fine aggregate and the important properties of shrinkage and plasticity in mortars.

An extensive study by H. V. Johnson[1] at the Bureau of Standards some years ago was a definite attempt to establish a logical basis for the acceptance of plastering sands. As stated previously, Johnson paid somewhat more attention to the strength of lime, gypsum and cement mortars proportioned arbitrarily in 1:2 and 1:3 mixtures by weight than seems justified by the purpose of the investigation, but his endeavor to use the Emley Plasticimeter on these mixtures yielded some interesting results. Out of 53 representative plastering sands he found that 26 passed the specifications for gypsum plastering sands then in effect,[2] while 27 did not. He also found that about as good results were obtained in plasticity with the sands not passing as with those passing. From a study of the plasticity figures given, however, one recognizes the urgent need of independent and more fundamental methods of investigating this property, and it is gratifying to note that determined efforts are now being made by Committee C-9 on Concrete and Concrete Aggregates and other agencies toward a solution of this problem.

In the present rather discouraging situation so far as a rating or selection of mortar and plaster sands is concerned, the thought occurred that the plastic mortar test described before this Society last year by Brickett[3] might be a means of distinguishing between good and poor sands, if only to the extent of indicating that some are more economical than others. Accordingly 35 sands were prepared for test in this manner as building sands. These were for the most part natural sands, just as received from time to time at the laboratory for the usual routine tests, except that all were scalped on a No. 8 sieve prior to test; that is, all of the sands in this series contained no material coarser than the No. 8 sieve size. The average quality of the lot was probably raised somewhat by including sands from six of the eight laboratories participating in the 1928–1929 series of cooperative tests of Committee C-1 on Cement;[4] a number of the sands were regraded from our own laboratory sand to give rather extreme ranges in grading; and a number were selected as being of rather unusual character, such as granulated blast-furnace slag, slag screenings, "haydite," alundum grits, stone dust, etc. The plastic mortar test was made in the following manner:

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[1] H. V. Johnson, "Sands for Lime, Gypsum and Cement Plasters," *Rock Products*, Vol. 28, p. 54 (1925).

[2] Tentative Specifications for Gypsum Plastering Sand, *Proceedings*, Am. Soc. Testing Mats, Vol. 24, Part I, p. 837 (1924).

[3] E. M. Brickett, "A Plastic Mortar Compression Test for Cement," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 432 (1928).

[4] See Report of Committee C-1 on Cement, *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part I, p. 258 (1929).

A quantity of uniform cement sufficient for the entire series was prepared in the usual manner and stored in an iron barrel. For any given sand a batch of mortar was prepared by taking 600 g. of this cement, 320 cc. of water (nominal $w/c = 0.8$), and enough of the sand to develop a given consistency (indicated by a flow of 45 on the 10-in. flow table). This batch was sufficient for making six 2-in. mortar cubes, two of which were for test at 3 days, two at 7 days and two at 28 days. Each sand was made up in this manner on at least two different days, so that at least two independent determinations were made of the quantity of each sand required to make up the plastic mortar. These determinations were usually in very close agreement, for the flow of the mortar was in all cases held within the limits 45 ± 5.

EXPERIMENTAL RESULTS FROM THE PLASTIC MORTAR TEST

The data of all the tests are given in Table I. Owing to the limited quantity of a number of the sands, it was necessary to make the unit weight determinations in a small cylindrical cup about 3 in. in diameter and 3 in. high. The accuracy is not as great as that of the standard method, but it is believed that the results are quite dependable. Fineness modulus, silt and color were determined by the standard methods. Absorption was determined by a titration method, recently proposed by the author.[1] It may be noted, however, that this method for absorption measurement fails on certain types of fine aggregates, particularly those having a spongy or vesicular structure; hence the omission of absorption values in a few cases. Ratios of cement to aggregate by weight and volume, and the quantities of cement per unit volume of mortar, were computed in the usual manner.

In trying to get something tangible out of the data of Table I, it should be borne in mind that we are not directly concerned with the absolute strengths of the mortars, which are in every case many times the strength required in a masonry mortar. Therefore, we have derived from the absolute strengths an "average relative strength" for each mortar as indicated in the table. We also have nothing to offer on the important properties of plasticity and shrinkage, and therefore about the only thing left as a basis for preference of one sand as compared with another is *economy*. In other words, those sands are rated highest which require the least amount of cement in a unit volume of mortar of definite consistency.

In Fig. 1 each sand is plotted on the axis of abscissas in the order of quantity of cement per unit volume of mortar, and other physical

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[1] J. C. Pearson, "A Simple Titration Method for Determining the Absorption of Fine Aggregate," *Rock Products*, Vol. 32, May 11, 1929, p. 64.

Table I.—Data of Tests of Fine Aggregates.

Fine Aggregate (Natural sand unless otherwise indicated)		Weight, lb. per cu. ft.	Fineness Modulus	Passing No. 50 Sieve, per cent	Silt, per cent	Colorimetric Test	Absorption, per cent	Sand-Cement Ratio		Cement, lb. per cu. ft. of Mortar	Compressive Strength of Mortar						
											lb. per sq. in.			Relative Strength			
Mark	Source or Type							By Weight	By Volume		3 days	7 days	28 days	3 days	7 days	28 days	Average
A	Morrisville, Pa	100	2.32	15	1.6	0	0.85	2.40	2.26	34.0	1460	2760	5800	85	86	100	90
B	Souderton, Pa	99	1.93	26	1.0	0	1.15	2.26	2.14	34.7	1200	2540	5470	70	79	94	81
C	Ottawa, Ill	109	1.74	26	0.2	0	0.03	2.84	2.45	30.8	1520	2660	5100	88	83	88	86
D	Cressona, Pa	102	2.34	22	5.2	0.5	0.75	2.50	2.30	33.1	2500	3700	6010	145	116	103	121
E	Cressona, Pa	99.5	2.34	15	1.6	0	1.05	2.42	2.28	33.6	1240	2660	5470	72	83	94	83
F	Cressona, Pa	106	2.43	22	8.4	0	0.25	2.48	2.20	34.2	2190	4000	7210	127	125	124	125
G	Towson, Md	102	2.06	26	6.6	1.5	0.35	2.33	2.14	34.8	1790	3530	6220	104	110	107	107
H	Hyattsville, Md	101.5	2.32	18	3.0	1.0	0.65	2.48	2.30	32.6	1510	2790	4980	88	87	86	87
I	Buffalo, N. Y	109.5	1.92	50	2.0	0	0.60	2.54	2.18	33.3	1940	3300	5930	113	103	102	106
J	Buffalo, N. Y	114	1.97	47	7.6	0.5	0.95	2.53	2.08	33.6	2130	3940	6580	124	123	113	120
K	Harrisburg, Pa	101	2.41	11	1.0	0	0.90	2.39	2.22	34.1	1430	2830	5660	83	88	97	89
L	Elgin, Ill	110.5	2.47	18	0.6	0	0.85	2.85	2.42	31.4	1540	2960	5600	89	92	96	92
M	Scranton, Pa	101	2.15	25	5.4	0	1.30	2.21	2.06	35.8	1650	3120	5330	96	98	92	95
N	Washington, D. C	101.5	2.20	21	3.0	1.0	0.70	2.32	2.14	34.8	1580	3080	5300	92	96	91	93
O	Sandt's Eddy, Pa	102.5	2.25	24	1.6	0	0.75	2.50	2.30	33.5	1520	3030	6070	88	95	104	96
P	Portland, Pa	106.5	2.43	24	4.2	0	0.80	2.67	2.35	32.9	1900	3860	6550	110	120	112	114
Q	Williamsport, Pa	99	1.34	56	6.4	0	1.00	1.88	1.78	39.0	1660	3240	5990	96	101	103	100
R	Williamsport, Pa	95	1.58	44	2.0	1.0	0.65	1.95	1.93	40.7	1480	2810	5610	86	88	96	90
S	Minneapolis, Minn	110.5	2.58	16	0.4	0	0.60	3.05	2.60	30.4	1420	2530	5460	82	79	94	85
T	Hereford, Md	102	2.20	25	6.6	0	1.10	2.16	1.98	37.1	1660	3150	5390	96	98	93	96
V	Same as "F," washed	101.5	2.69	14	1.0	0	0.25	2.58	2.38	32.4	1540	3010	5750	89	94	99	94
W	Attica, Ind	106	2.95	6	0.6	0	0.95	2.99	2.65	30.4	1570	3140	5980	91	98	103	97
X	Shillington, Pa	101	2.60	23	13.0	4+	0.70	2.29	2.13	35.8	2020	4200	6860	117	131	118	122
Z	Allentown, Pa	95.5	2.30	29	9.0	0	1.10	1.96	1.93	38.6	2030	3910	6610	118	122	114	118
AA	Granulated Blast-Furnace Slag	45	2.54	27	...	0	....	0.66	1.37	52.1	1610	2750	4800	93	86	82	87
BB	Crushed Alundum	83.5	3.39	7	2.0	0	....	1.81	2.04	41.4	1360	2770	5820	79	87	100	89
CC	Crushed Blast-Furnace Slag	96	2.27	34	15.0	0	....	1.79	1.75	41.4	2580	4410	7810	150	138	134	141
DD	Stone Dust	115	1.20	65	23.0	0	....	1.78	1.45	41.8	1920	3440	6020	111	107	103	107
FF	"A" regraded (No. 16 – No. 100)	97.5	1.54	54	0	0	0.85	1.99	1.92	37.6	1430	2660	5140	83	83	88	85
GG	"A" regraded (No. 8 – No. 100)	98.5	2.32	12	0	0	0.85	2.39	2.28	33.9	1410	2670	5220	82	83	90	85
KK	Cow Bay, L. I	104.5	2.23	24	3.6	0	0.27	2.51	2.26	33.6	2220	3720	5840	129	116	100	115
LL	Haydite	59	2.89	16	4.6	0	....	1.19	1.90	38.9	2390	4370	6720	139	137	115	130
MM	Springfield, Vt	105	2.20	27	4.4	1.0	1.05	2.67	2.38	32.3	1710	2780	5010	99	87	86	91
NN	"A" regraded (No. 50 – No. 100)	93	0.93	100	0	0	0.85	1.70	1.71	40.1	1480	2650	4760	86	83	82	84
QQ	"A" with added silt	106	1.85	28	9.0	0	0.85	2.42	2.14	33.6	1760	3080	5550	102	96	95	98

properties are plotted on its ordinate at different levels to suitable scales. From the trend of any group of corresponding points, it can be determined whether the property represented by this group bears any relation to the "cement factor," or to the other properties.

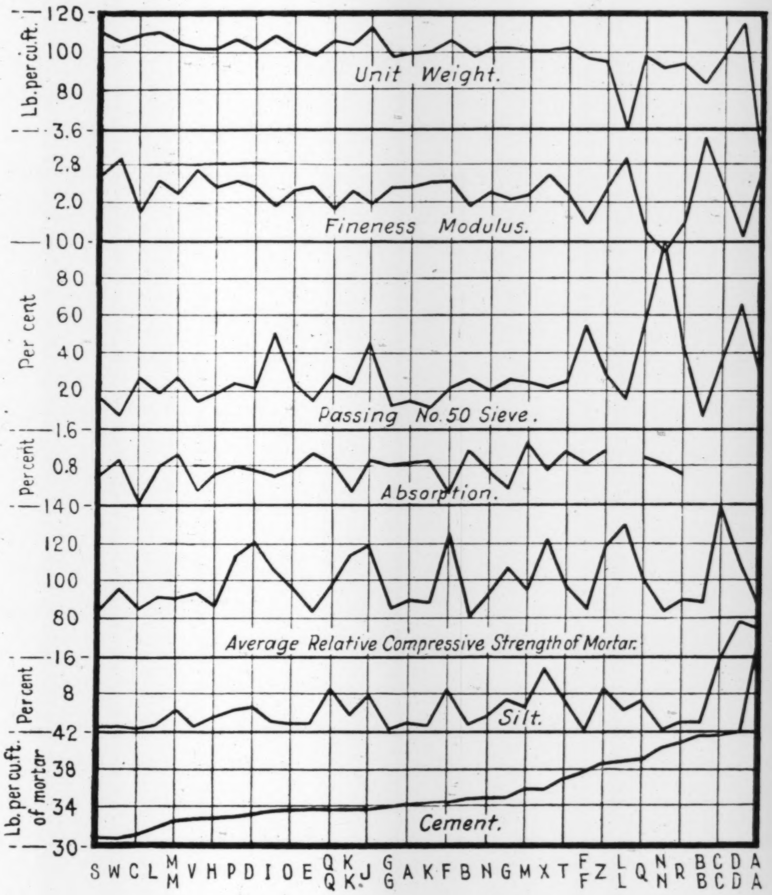

FIG. 1.—Data of Tests of Fine Aggregates.

A first glance at Fig. 1 is somewhat discouraging in that we have apparently little relation between the various properties of the fine aggregate. In most of the groups there is a ragged saw tooth arrangement, which suggests either that there is no relation at all, or that there are other factors than the cement factor upon which the points are more dependent for their position. The latter is certainly true to some extent, as we shall endeavor to point out.

It will be noted that there is a slight but definite trend upward in the curves for silt, relative strength, absorption, and percentage of aggregate passing the No. 50 sieve. There is a slight but definite trend downward in the curves for unit weight and fineness modulus. Probably all of these trends are as we should expect them to be. For example, in the case of absorption, the effect of this is to rob the mix of some of its water. The high absorptions of certain aggregates would therefore materially cut down the amounts of those aggregates required to bring the mortars to the specified consistency, and therefore the cement content of these mortars would be higher, other things being equal. Nevertheless it is probable that other factors, such as grading, would have a much more pronounced effect on the quantity of cement required.

One of the most interesting things brought out by the study of these sands is the effect of silt. If we examine closely the curves for silt and relative strength in Fig. 1, we notice quite a remarkable tendency toward parallelism in these curves, especially on the right-hand side of the diagram. Nearly all the sands which give exceptionally high strengths are high in silt. The probable explanation seems to be that the high silt behaves somewhat like a powdered admixture, and has a capacity for absorbing or taking unto itself considerable water from the mix, and in this manner has a much greater effect, in extreme cases, than the absorption of the fine aggregate itself. We cannot, of course, advocate the unrestricted use of high silt sands for mortar and plaster, but if it is known that a high silt sand also gives a relatively high strength in tests such as those here reported, it should logically qualify as a good mortar or plaster sand.

As might be expected there is a marked inverse relation between the fineness modulus and the percentage of fine aggregate passing the No. 50 sieve. The reason for including the latter was to show how frequently and how far the 20-per-cent limit in one of the specifications referred to (Specification C 35 – 25) is exceeded in what we believe to be perfectly good mortar sands. If we accept the first 24 of these sands as suitable from the standpoint of an economical cement factor we note that only nine of these come within the specified limit.

We will now discuss the results from a few of the individual sands, to bring out a little more definitely some of the advantages of a testing procedure based on a fixed cement-water relation, rather than upon a fixed cement-aggregate relation.

Sands A, FF, GG, NN and QQ are all modifications of our laboratory sand ranging in fineness modulus from 0.93 to 2.32. Note how closely the average relative strengths of FF, GG and NN agree; A is slightly higher with a small silt content, and QQ is considerably higher with a large silt content. FF and NN are immediately ruled out, not on the basis of high fineness, but because the cement factors are high.

Sand F is a sharp well-graded sand of high silt content. When its remarkably high strength was first noted, a portion of this sand was washed and tested as sand V. The relative strengths of the two are 125:94, whereas the cement factor for F is only slightly over 5 per cent higher than that for V. In a case of this sort we are undoubtedly testing the high silt sand at a net water-cement ratio considerably lower than the nominal ratio. We have no means at present of knowing how much to allow for the water requirement of the silt, but with the refinement which seems possible in the plastic mortar test, we could determine by trial the difference in nominal water-cement ratio required to bring sands with and without silt to the same mortar strengths. This would give us a means of attack, but we should not lose sight of the fact that silt is at best an uncertain element. The problem is a more definite one in relation to inert powdered admixtures, and those who are interested in the study of admixtures will find the plastic mortar test a useful tool.

Sands I and J are of unusual grading in consisting largely of coarse and fine particles with few intermediate size grains. While nearly 50 per cent of these sands passes the No. 50 sieve, they have a very satisfactory cement factor in these tests. Note the high unit weights of these sands, also that the relative strength of J, a high silt sand, is 120.

Sands T, Z and MM are sands which in the present state of our knowledge we could not accept as satisfactory sands for concrete. All of them would probably be satisfactory for ordinary mortar and plaster, although sand T would be the least economical from the indications of these tests. Sands T and MM have both been considered the cause of disintegration of concrete in which they were used; sand Z is a friable, light-weight sand of the type we are warned against (but not adequately informed about) in the Society's Tentative Specifications for Concrete Aggregates (C 33 – 28 T).[1]

It is quite remarkable in a miscellaneous collection of sands as large as this that so many should be free from organic impurities and that only one should have a color value greater than 1.5. Sand X gave a dark coffee color in the sodium hydroxide test but nevertheless developed a relative strength of 122. Either the organic matter was of such nature as not to interfere with the hardening of the cement, or else its effect was offset by the virtue in 13 per cent of silt. However, the usefulness of the plastic mortar test in checking the indication of the colorimetric test is one of its recommendations. Since the tests reported herein were made, a sample of sand was received by the laboratory which gave a color of 4 in the sodium hydroxide test. In the plastic mortar test (with the cement used in this series) the 3-day strength of the mortar from this sand was 634 lb. per sq. in., that is, about 50 per cent of the lowest strength recorded in Table I.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 822 (1928); also 1928 Book of A.S.T.M. Tentative Standards, p. 238.

A few remarks might be of interest in regard to the "unusual" aggregates, AA, BB, CC, DD and LL. Aggregate AA is an extremely light, friable granulated slag, so feeble that it was easily broken up in the test for weight per unit volume. It is quite remarkable that the strength of mortar made with this material was not far from the average strength. Aggregate BB is crushed alundum which under the microscope is seen to have a very open structure. Why this material does not absorb the water from the mix and give a high strength like aggregate LL (haydite), for example, is not clear. Haydite gave the highest strength (with one exception) of all the materials tested. A part of this is undoubtedly due to high absorption, a part may be due to good mechanical bond. It is to be noted that any of the fine aggregates that give a high relative strength in these tests are entitled to a lower cement factor. One way of looking at this is to assume that the corresponding mixtures have been penalized on water-cement ratio; another is that they are richer than need be to produce the estimated strength. The chief objection to the crushed fine aggregates is their harshness, and this might debar them in spite of all their other virtues. The crushed blast-furnace slag, aggregate CC, gave the highest of all mortar strengths. How much of the excess can be explained by absorption, mechanical bond and high "silt" we do not know, but previous investigations have shown that the dust in slag screenings is not in the same class as inert powdered admixtures. There is little doubt of chemical action in this case. Aggregate DD is the waste material from a stone working plant and was obtained as a sample of one type of aggregate used in a concrete products plant. The plastic mortar test shows quite definitely that one reason for not using such material for general purposes is the large amount of cement required.

CONCLUSIONS

What seems to the author to be established from general experience, and confirmed by the references cited and the test data offered herein, is that almost any materials worthy the name of fine aggregate can be used satisfactorily as aggregate for ordinary mortar and plaster; but some, of course, are better than others.

The specifications that now exist are indicative of opinions only in regard to satisfactory aggregates; if taken literally they will rule out many perfectly good aggregates.

Rather than set definite gradation limits on an aggregate (except possibly maximum size) for mortar and plaster, it would seem better to guard against the use of dangerous or uneconomical aggregates by means of the plastic mortar test as described. So far as developed, this test applies only to cement mortars, but it may be found applicable to other types.

Before complete specifications for fine aggregates for mortar and plaster can be written, general relations will have to be established between the characteristics of the aggregates and the properties of plasticity and volume change. These are the most important properties of ordinary mortar and plaster, and they are also the ones that are quantitatively beyond our reach at the present time.

FINE AGGREGATE IN BITUMINOUS MIXTURES

By Hugh W. Skidmore[1]

Synopsis

The fine aggregate portion of bituminous mixtures may vary from a small amount (10 to 20 per cent) in very coarse, open mixes, to 100 per cent in fine sheet mixes, and, consequently, it may serve primarily as a void filler in coarse mixtures until the amount of fine material exceeds in bulk the coarse, whereupon the situation is reversed and the coarse aggregate reduces voids in the fine; or it may constitute the whole mineral structure.

In all filled mixtures, there are two distinct aggregates that require separate treatment in test and design.

1. Fine aggregate exclusive of filler—the normal sand portion; and
2. Filler.

They have many properties and functions in common but also require individual or separate consideration before they are combined.

The important properties or characteristics of fine aggregates requiring recognition and test are, in the case of unfilled fine aggregate (sand), (*a*) voids in dry state, (*b*) grading, (*c*) specific gravity, (*d*) porosity and rugosity, (*e*) shape of grains, (*f*) stabilizing ability, and (*g*) hardness and toughness. In the case of filler, these characteristics include (*a*) void-reducing ability, (*b*) stabilizing ability, (*c*) fineness, (*d*) specific gravity, (*e*) shape of particles, and (*f*) porosity.

The significance of the properties of aggregates are inseparably associated with mixture performance and many of the final tests must be made upon the mixture itself. Composite aggregates, exclusive of filler, require preliminary examination to ascertain the optimum combination with respect to practicality, voids, grading and specific gravity. Then filler must be added in varying amounts for further study. The mixtures are finally tested for compressibility (mixture voids); stability (resistance to shear) at 140° F.; absorption of moisture; and denseness. No two aggregates or fillers behave exactly alike in mixtures, hence criteria cannot be established for a given kind or class. Certain porous aggregates have distinct advantages over non-porous ones both with respect to stability and durability in service. Likewise certain fillers may be excellent void reducers and yet produce poor compressibility of mixture while still others impart marked increase in stability.

Fine aggregates cannot be intelligently accepted or rejected until a thorough study of mixture performance has been made.

The subject of fine aggregate in bituminous mixtures immediately divides itself into two distinct sections requiring somewhat different treatment in discussion:

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1. Fine aggregate, exclusive of filler; and
2. Filler.

Although the primary function of both is the reduction of voids both as to amount and size, the filler performs a special function in stabilizing the mixture. In fine mixtures such as sheet asphalt, the mineral aggregate is 100 per cent fine, but in bituminous concrete it may vary from only 10 or 15 per cent in very coarse, open mixtures, to 80 per cent or more in stone-filled sheet mixtures.

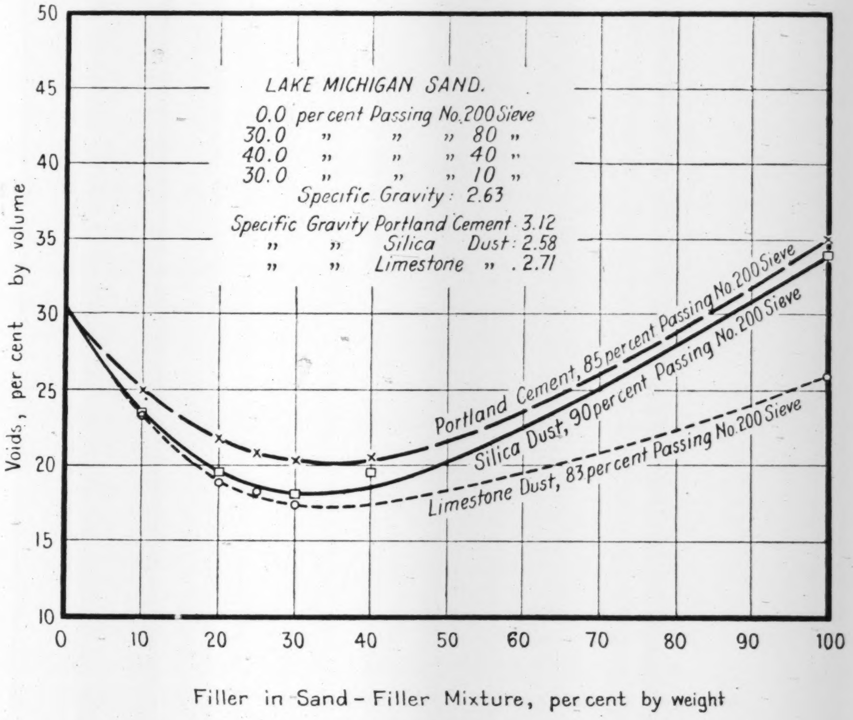

FIG. 1.—Results of Tests for Voids in a Given Sand with Special Types of Filler.

Obviously, then, its function with respect to reducing voids in coarse aggregate may be gradually modified until a small amount of coarse aggregate actually reduces the voids in the fine aggregate as in the case of stone-filled mixtures. Fortunately, a general conception of sheet asphalt, with respect to sand-filler combinations, gives also similar knowledge of the combined aggregate in even a coarse aggregate type of mixture, the same laws relative to voids, bitumen content and stabilizing value of sand and filler holding true for all, with special consideration being given to coarse aggregate properties such as hardness, toughness, etc.

When one bears in mind the fact that fine aggregate in addition to filling voids in coarse aggregate and greatly increasing its impermeability, also reduces harshness of the mixture and renders it more workable and compressible, he is prepared to study the behavior of fine aggregate, that is, the sand and filler, as a separate entity, realizing that it behaves in like manner when combined with coarse aggregate. In other words, when considering filled mixtures, we simply move up or down the scale as the maximum size of aggregate decreases or increases.

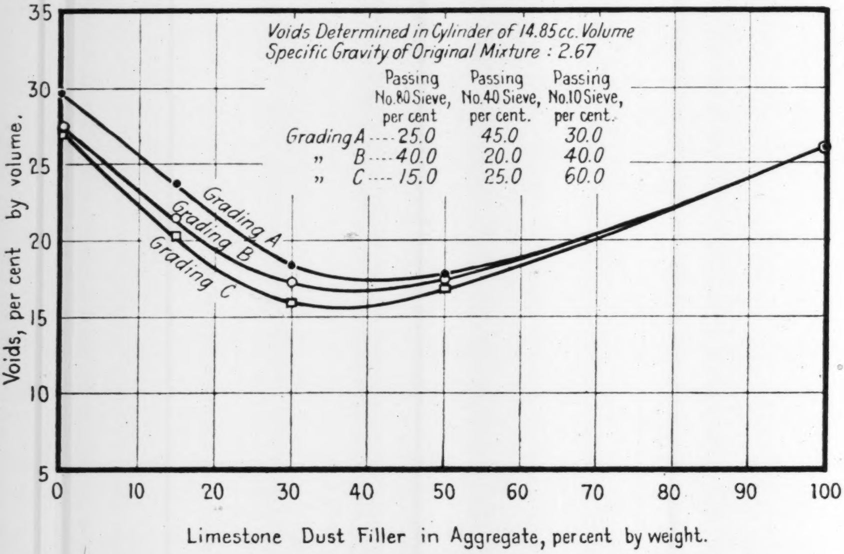

FIG. 2.—Showing Difference in Voids with Variations in Grading of a Given Sand in Combination with a Given Filler.

VOIDS

As the result of the work of a number of investigators, the determination of aggregate voids in comparatively heavy metal containers capable of receiving repeated, heavy blows and employing dry aggregate packed to refusal, has proved to be the most reliable method. This method gives lower voids than any other method so far investigated and gives results that are entirely reliable for use in designing mixtures, the results checking field practice.

Typical results on sand-filler combinations are shown in Fig. 1. Curves of this sort serve as the basis of mixture design. It will be seen that the three fillers give different results with a given sand, and this is typical. Because of this difference in fillers, it is essential to examine each one proposed for use.

It is obvious that the filler producing the lowest voids, other things being equal, will be the most economical from the standpoint of asphalt required, and unless the difference in cost of filler offsets this, void-reducing ability may govern in the purchase. This holds true unless there is considerable difference in workability and compressibility of mixtures employing the various fillers examined.

Fine aggregates exclusive of filler also vary greatly with respect to voids and may influence workability and compressibility very measureably. Manipulation of proportions of a composite sand may have great influence upon mixture economy and quality of finished work by reducing voids, rendering mixtures more workable, etc. The first step in selection of aggregate then becomes a study of fine aggregate available exclusive of fillers, then in combination with available fillers. Selection results from a consideration of:

1. Voids;
2. Workability and compressibility of mixtures under field conditions, which can be simulated in the laboratory;
3. Stability of resulting mixture;
4. Moisture absorption; and
5. Durability of aggregate, that is, resistance to abrasion, crushing, etc., which is worthy of consideration but is ordinarily of secondary importance, since quite porous, soft aggregates have proved their merit in bituminous mixtures. They are usually inferior only in the case of large sizes in concretes or macadam. In some respects, soft, porous fine aggregates are substantially superior to sharp, hard ones.

Figure 2 illustrates the difference in voids with variation in grading of a given sand in combination with a given filler. This figure will also be used in the discussion of compressibility and stability.

WORKABILITY

Workability cannot readily be measured mechanically in the laboratory and is really very largely dependent upon observation during mixing and placing operations. The temperature of the mixture and the conditions of the atmosphere affect workability. Certain aggregates produce poor workability due to the grading, surface texture, angularity, etc., and certain fillers are either beneficial or otherwise. Compressibility is related to workability but not necessarily in a direct ratio. For example, one filler may increase workability and decrease compressibility as compared with another. But poor com-

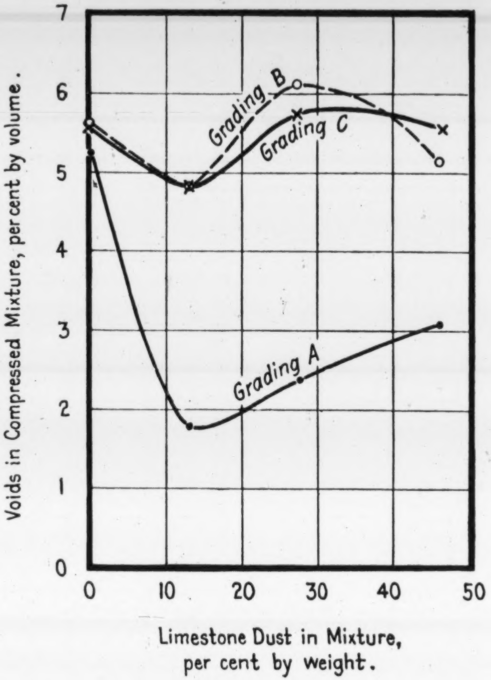

FIG. 3.—Showing Effect of Grading of Fine Aggregate on Compressibility.

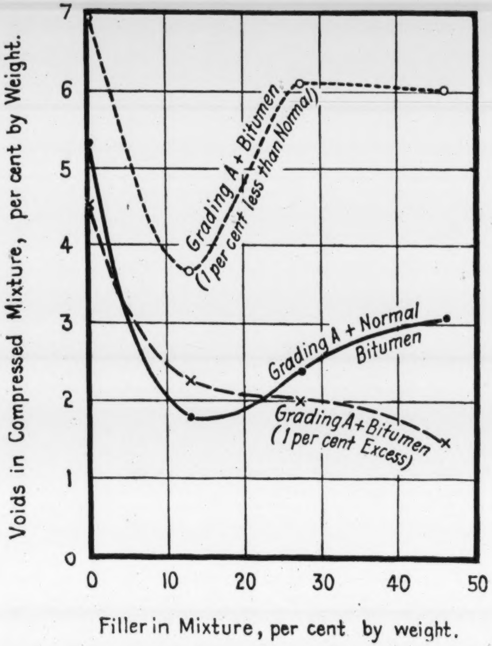

FIG. 4.—Showing Effect of Variation of Bituminous Content on Compressibility of Bituminous Mixtures.

pression follows poor workability purely as a mechanical function. The experienced laboratory technician can detect good or bad mixtures with respect to workability during their preparation in the laboratory.

COMPRESSIBILITY

Compressibility is one of the most important properties of any bituminous mixture, as it affects the durability of the pavement through denseness (resistance to moisture), ravelling, cracking and surface roughness (such as choppy waves). If a mixture cannot ordinarily be compressed to within approximately 95 per cent of its theoretical maximum density, it may prove to be unsatisfactory, and if the compression falls much below this, early failure is inevitable.

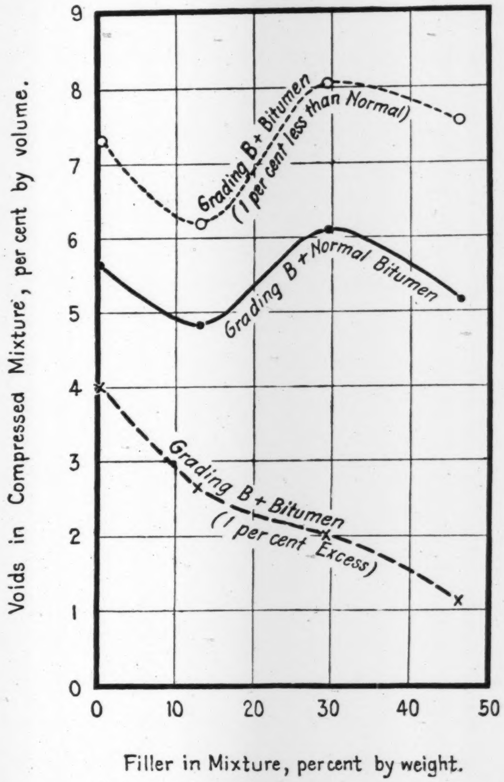

FIG. 5.—Showing Effect of Variation of Bituminous Content on Compressibility of Bituminous Mixtures (Inferior Sand Grading).

The effect of grading of fine aggregate upon compressibility is shown in Fig. 3. Mixtures were made up with various amounts of a given filler and the fine aggregate gradings of Fig. 2, using the same asphalt for all in an amount required to fill voids, with the result that grading A produced excellent compressibility while the others were correspondingly poor. It is also obvious that the amount of filler has an effect upon the compressibility of the mixture, in this case reaching a maximum at about 15 per cent and gradually increasing mixture voids as the amount of filler increases. It is significant that a decrease in amount of filler reduces compressibility much more rapidly than does an increase. While other aggregates will not agree exactly with this case it is typical of such a study and clearly indicates the importance of this property.

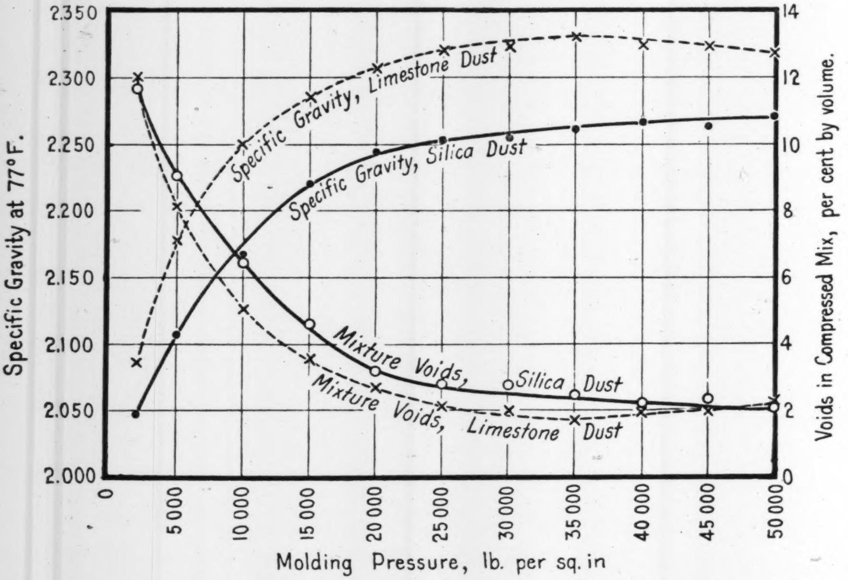

FIG. 6.—Showing Relation Between Specific Gravity and Compressibility of Bituminous Mixtures to Intensity of Molding Pressure.

Compressibility is also affected by the amount of bitumen employed in the mixture as is shown in Fig. 4. Less bitumen than is required to fill voids reduces compressibility and, except in a few cases of peculiarities of certain fillers or fine aggregates, an excess of bitumen offers no advantage except for high filler content well beyond the amounts ordinarily used in paving practice. Figure 5 illustrates this point in the case of an inferior sand grading, Grading B of Fig. 2. Similar data for Grading C duplicates Grading B very closely.

The relation between specific gravity, compressibility (mixture voids) and loads applied (vertical pressure) in forming test cylinders 1 in. in diameter is shown in Fig. 6, the materials being taken from supplies in use. It will be noted that there is practically no gain beyond 25,000 lb. per sq. in. Pressure of this type of 5000 lb. per sq. in. is equivalent to rolling with 10 to 12-ton 3-wheeled rollers in the field.

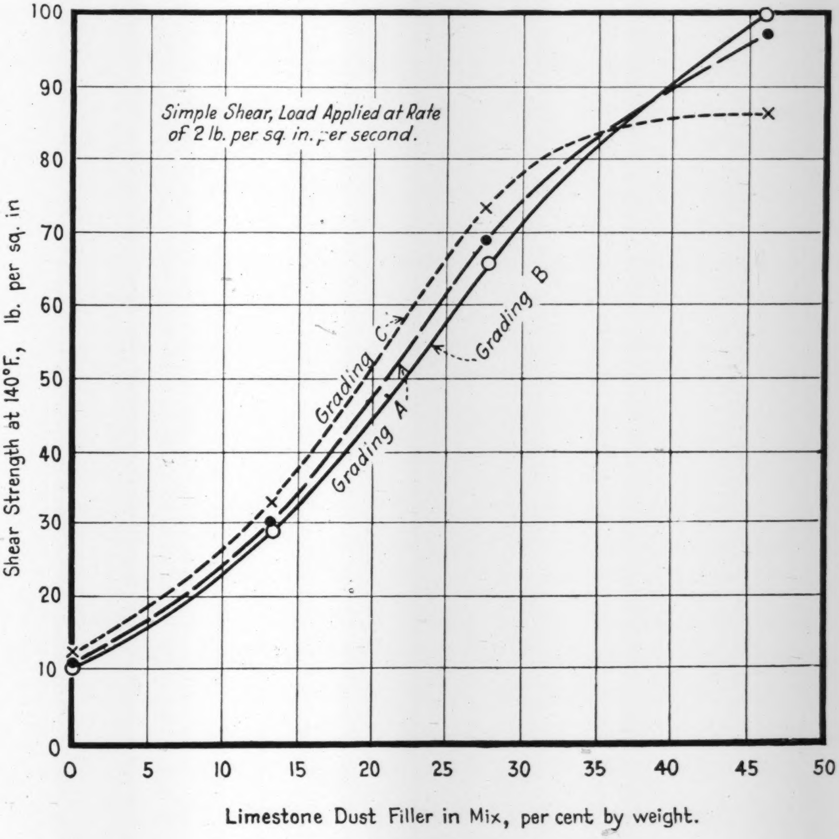

FIG. 7.—Relation of Shearing Strength to Filler Content of Mixtures of Several Gradings with Asphalt Cement Just Filling Voids in Aggregate.

NOTE.—Gradings are those shown in Fig. 2. Asphalt cement: Mexican asphalt of 40 penetration. See Table I.

STABILITY (SHEAR STRENGTH AT 140° F.)

A detailed description of a test for stability is given in an earlier article by the author.[1]

Stability is an important property of mixtures. Tests are made at 140° F. as this represents about the maximum pavement tempera-

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[1] H. W. Skidmore, *Roads and Streets*, January, 1929.

TABLE I.—RESULTS OF STABILITY TESTS ON MIXTURES OF SEVERAL GRADINGS WITH ASPHALT CEMENT JUST FILLING VOIDS IN AGGREGATE.

Data plotted in Fig. 7.

Voids in Aggregate, per cent	Bitumen to Fill Voids, per cent[a]	Theoretical Specific Gravity	Specific Gravity of Cylinder, 77° F.[b]			Shearing Strength at 140° F.,[b] lb. per sq. in.			Voids in Compressed Mix, per cent
			Minimum	Maximum	Average	Minimum	Maximum	Average	
				GRADING A					
29.7	14.3	2.190	2.070	2.080	2.075	8.9	9.9	9.5	5.25
23.9	11.0	2.288	2.241	2.255	2.247	27.7	30.5	29.2	1.79
18.5	8.2	2.380	2.320	2.329	2.323	67.2	72.2	69.8	2.39
17.9	7.9	2.396	2.317	2.327	2.322	92.3	110.3	97.4	3.09
				GRADING B					
27.5	13.0	2.226	2.090	2.111	2.101	8.9	10.1	9.7	5.62
21.5	9.7	2.327	2.211	2.219	2.215	27.3	30.3	28.7	4.82
17.2	7.5	2.402	2.248	2.261	2.255	58.9	73.4	65.7	6.12
17.4	7.6	2.405	2.273	2.290	2.281	95.5	106.0	100.6	5.16
				GRADING C					
27.1	12.8	2.231	2.069	2.119	2.102	11.2	12.1	11.7	5.58
20.3	9.1	2.346	2.226	2.237	2.233	30.3	36.6	32.9	4.82
16.0	6.9	2.420	2.270	2.300	2.280	70.0	80.2	73.4	5.78
16.9	7.4	2.413	2.276	2.284	2.279	77.9	103.3	86.6	5.55

[a] Specific gravity of bitumen 1.053.

[b] Two-inch cylinders molded at 350° F. under pressure of 5000 lb. per sq. in.

TABLE II.—RESULTS OF STABILITY TESTS ON MIXTURES OF FINE AGGREGATE WITH VARYING AMOUNTS OF ASPHALT CEMENT.

Data plotted in Fig. 8.

Voids in Aggregate, per cent	Bitumen, per cent		Theoretical Specific Gravity	Specific Gravity of Cylinder, 77° F.[a]			Shearing Strength at 140° F.,[a] lb. per sq. in.			Voids in Compressed Mix, per cent
	Required to Fill Voids	Used		Minimum	Maximum	Average	Minimum	Maximum	Average	
					NORMAL ASPHALT CONTENT					
29.7	14.3	14.3	2.190	2.070	2.080	2.075	8.9	9.9	9.5	5.25
23.9	11.0	11.0	2.288	2.241	2.255	2.247	27.7	30.5	29.2	1.79
18.5	8.2	8.2	2.380	2.320	2.329	2.323	67.2	72.2	69.8	2.39
17.9	7.9	7.9	2.396	2.317	2.327	2.322	92.3	110.3	97.4	2.27
					ASPHALT 1 PER CENT IN EXCESS					
29.7	14.3	15.3	2.187	2.075	2.091	2.088	5.1	8.6	7.1	4.52
23.9	11.0	12.0	2.263	2.195	2.218	2.212	15.3	19.7	18.3	2.25
18.5	8.2	9.2	2.353	2.282	2.314	2.306	35.7	42.0	39.4	2.00
17.9	7.9	8.9	2.341	2.297	2.310	2.306	81.1	91.9	88.6	1.49
					ASPHALT 1 PER CENT LESS THAN NORMAL					
29.7	14.3	13.3	2.246	2.084	2.091	2.088	6.4	8.9	7.9	6.95
23.9	11.0	10.0	2.317	2.223	2.243	2.232	21.7	29.9	25.4	3.67
18.5	8.2	7.2	2.399	2.244	2.261	2.253	43.9	51.6	48.2	6.10
17.9	7.9	6.9	2.428	2.278	2.293	2.282	81.1	89.0	85.5	6.02

[a] Two-inch cylinders molded at 350° F. under pressure of 5000 lb. per sq. in.

ture during hot weather when mixtures are most apt to distort. It has been found to be a very valuable test for both laboratory investigation and control during construction. Stability is materially affected by such factors as:

1. Variations in grading of aggregate;
2. Shape, surface texture, porosity and rugosity of aggregate;

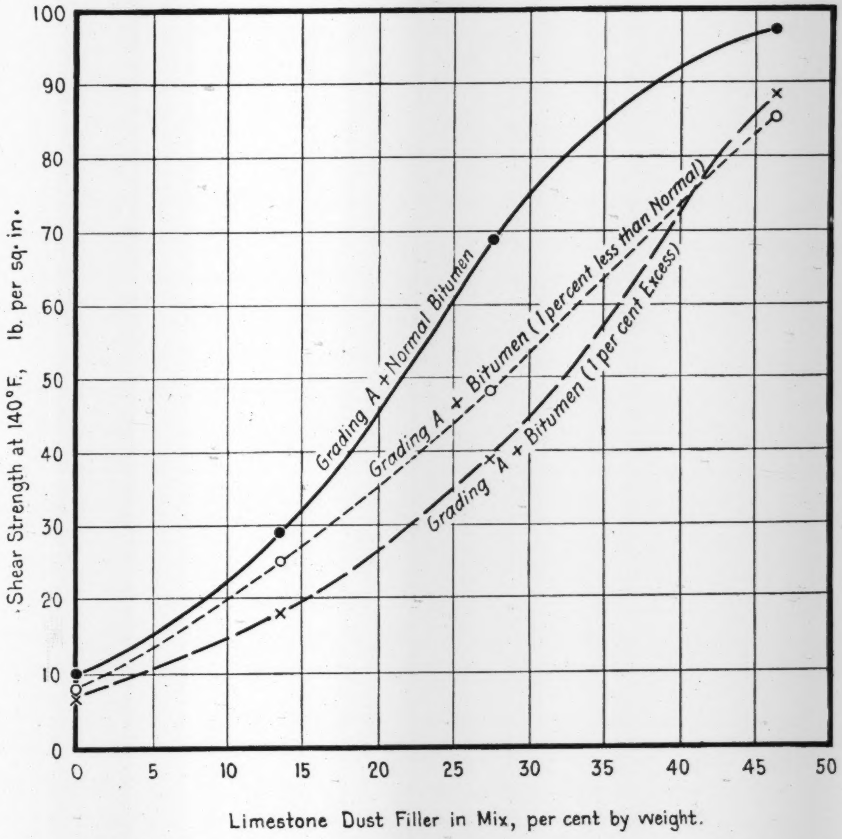

FIG. 8.—Showing Effect of Shearing Strength of Varying Amount of Asphalt Content.

NOTE.—Grading shown is Grading A of Fig. 2. See Table II.

3. Filler fineness, shape of grains, etc.;
4. Nature and consistency of the bitumen;
5. Denseness of mixture;
6. Uniformity of mixture which is directly associated with manufacture;
7. Poor workmanship in construction; and
8. Temperature of the test.

Figure 7 and Table I show the stabilities of the mixtures made from aggregates of the gradings shown in Fig. 2. Here we find nicely illustrated the fact that stability bears a rather constant relation to amount of filler in the aggregate. Figure 8 and Table II clearly show the falling off in stability when the bitumen is substantially either more or less than the amount required to fill aggregate voids.

TABLE III.—RESULTS OF TESTS ON COMBINATIONS OF FINE AGGREGATE AND FILLER

Fine Aggregate	Filler	Specific Gravity of Aggregates	Voids in Aggregate, per cent	Bitumen Required, per cent	Specific Gravity of Mixture		Voids in Compressed Mixture, per cent	Apparent Porosity of Fine Aggregate, per cent	Composition of Mixture per cent by weight					Shearing Strength at 140° F., lb. per sq. in.	Absorption per cent		
					Theoretical Maximum	Compressed Cylinders			Bitumen	Filler	80	40	10		24 hr.	48 hr.	96 hr.
Granite	Limestone	2.636	22.2	10.22	2.284	2.207	2.7	0.16	10.22	17.96	17.96	32.32	21.54	52.3	0.5	0.8	0.9
Ottawa Sand	Limestone	2.659	16.6	7.30	2.392	2.165	9.5	....	7.3	18.54	18.54	33.37	22.25	26.4	2.6	2.7	3.1
Lake Michigan Sand	Limestone	2.681	22.2	10.07	2.319	2.226	2.8	....	10.07	18.0	18.0	32.36	21.57	20.0	0.4	0.5	0.7
Wise County, Tex., Limestone	Limestone	2.688	17.5	7.67	2.402	2.310	3.8	1.5	7.67	18.47	18.47	33.23	22.16	119.2	0.3	0.4	0.7
Uvalde County, Tex., Leached Rock-Asphalt	Limestone	2.681	23.8	10.92	2.293	2.222	3.1	5.0	10.92	17.82	17.82	32.07	21.37	107.0	0.4	0.6	0.8
Granite	Silica	2.612	18.6	8.43	2.322	2.042	12.1	....	8.43	18.31	18.31	32.97	21.98	44.4	1.6	2.3	3.6
Granite	Silica	2.612	18.6	8.43	2.297	2.082	9.4	....	9.43	18.11	18.11	32.61	21.74	32.1	1.3	1.9	2.7
Ottawa Sand	Silica	2.634	16.9	7.51	2.367	2.103	11.1	....	7.51	18.5	18.5	33.3	22.19	26.0	1.2	1.3	2.1
Ottawa Sand	Silica	2.634	16.9	7.51	2.295	2.145	6.5	....	8.51	18.3	18.3	32.94	21.98	17.0	0.7	0.9	1.6
Lake Michigan Sand	Silica	2.656	19.3	8.65	2.346	2.116	9.7	....	8.65	18.27	18.27	32.89	21.92	39.5	1.0	1.3	2.0
Lake Michigan Sand	Silica	2.656	19.3	8.65	2.316	2.153	7.0	....	9.65	18.07	18.07	32.53	21.68	31.3	0.5	0.6	1.0
Wise County, Tex., Limestone	Silica	2.663	19.6	8.78	2.347	2.192	6.6	....	8.78	18.24	18.24	32.85	21.89	65.9	0.7	0.9	1.5
Wise County, Tex., Limestone	Silica	2.663	19.6	8.78	2.331	2.243	3.8	....	9.78	18.04	18.04	32.48	21.66	57.0	0.3	0.5	0.7
Uvalde County, Tex., Leached Rock-Asphalt	Silica	2.656	23.1	10.65	2.285	2.100	8.1	....	10.65	17.87	17.87	32.17	21.44	72.4	1.2	1.5	2.1
Uvalde County, Tex., Leached Rock-Asphalt	Silica	2.656	23.1	10.65	2.276	2.174	4.5	....	11.65	17.67	17.67	31.81	21.20	64.4	0.5	0.7	0.9

ABSORPTION

Absorption, while an important property of the mixture, is not necessarily determined in control work, since mixture voids serve as an accurate index of mixture absorption. It is quite essential in original investigation of materials. Some materials, such as certain clays, might be proposed for use as fillers and might show good stability and low mixture voids, but when mixtures containing them are subjected to moisture, they disintegrate rapidly, due to emulsification of the bitumen.

Arbitrary tests on fillers for emulsifying properties have been proposed by some as a quality test. They are apt to be very misleading unless performed by one thoroughly familiar with the subject of emulsions. Such tests are not only unreliable but unnecessary, since the absorption test is a certain detector of unsuitable materials in this respect.

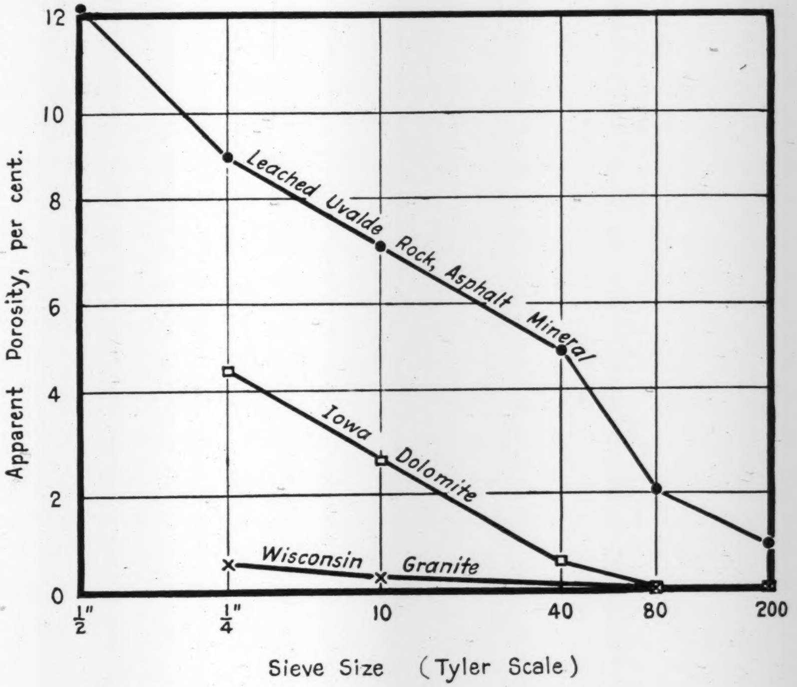

FIG. 9.—Typical Porosity Curves for Three Classes of Material.

Table III gives the results of a study of five distinctly different fine aggregates in combination with a given amount of two fillers widely used in practice, as follows:

Fine Aggregates

1. Crushed red Wisconsin granite, a very hard, tough non-porous material;
2. Ottawa sand, very sharp and non-porous;
3. Lake Michigan sand, widely used in asphalt paving work;
4. Crushed limestone from Wise County, Tex. A soft material of moderate porosity; and
5. Crushed limestone, from the region of lime-rock asphalt, Uvalde County, Tex., from which the bitumen has supposedly leached out by exposure. Very porous and rather soft, though not as easily crushed as No. 4.

Fillers

1. Limestone dust of excellent quality produced by the Black-White Lime Co. of Quincy, Ill. (83 per cent passing a No. 200 sieve); and
2. Silica dust of excellent quality produced by The Toledo Vitrified Brick Co., Toledo, Ohio (90 per cent passing a No. 200 sieve).

Bitumen

Mexican asphalt produced by the Standard Oil Co. of New Jersey, of 61 penetration at 77° F.

All mixtures were prepared and tested in strict accordance with standardized laboratory procedure as described in detail in the paper previously cited.[1] Apparent porosity was determined by calculation from the displacement in kerosine or any fairly stable liquid which wets the surface freely, in a Le Chatelier specific gravity bottle at 77° F. The sample was separated into various sizes (passing No. 200, No. 80, No. 40, No. 10, etc., sieves) and the readings taken for each size independently. Specific gravity of the whole was taken as that of the portion passing the No. 200 or No. 80 sieve (these are usually the same). Particles No. 80-size and finer show very low porosity even in the case of very porous materials. Typical porosity curves on three classes of material are shown in Fig. 9; a very porous material with pores extending into very small sizes; a moderately porous dolomite with pores confined to larger sizes; and a non-porous granite.

The table includes data on voids, compressibility, stability or shear strength, and absorption.[2]

Voids.—The silica dust produced lower voids than the limestone dust in the case of granite, Lake Michigan sand and Uvalde County limestone, with the limestone dust superior in the other two cases.

Compressibility.—The limestone dust produced much more compressibile mixtures in all cases. An increase in bitumen in the case of silica dust increased compressibility for all aggregates but also caused a falling off in stability of from 8 to 12 lb. per sq. in.

Limestone dust produced low mixture voids in all cases except Ottawa sand. The extreme sharpness of sand grains in this case is thought to be the reason for poor compressibility.

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[1] H. W. Skidmore, *Roads and Streets*, January, 1929.

[2] Since this paper was prepared, a study has been made of mixtures employing the same silica dust used in these experimental mixtures, in comparison with a limestone dust widely used in the same district as the silica. These two fillers were combined with a lake sand of excellent quality, common to the territory, in different proportions and the mixtures were then modified to make proper allowance for the difference in volume-weight of the two fillers.

The results obtained plainly show that the two fillers give equivalent mixtures with respect to vital characteristics such as stability and absorption, with the silica dust only slightly less compressible and at the same time productive of slightly greater yield. Thus it is seen that what may hold for one combination of aggregates may not hold for another.

Stability (Shear Strength.)—It is very obvious that increasing bitumen beyond the amount required to fill voids reduced stability. Silica dust mixtures gave lower stability for all aggregates except Lake Michigan sand.

There is a vast difference in stability value of different aggregates. Porous aggregates, even the very soft limestone, show much higher values than the sands, with crushed granite intermediate. The rugosity of the granite, although the granite was of non-porous structure, has a decided effect upon stability due to offering more points of contact. The water-worn lake sand naturally is low in value, with only the Ottawa sand-silica combination showing lower strength.

The unusually high stability of the Texas limestone is borne out in practice by the great durability and stability of limerock asphalt pavements, such as those produced from Texas and Alabama deposits. These mixtures, properly compounded, even when laid very thin (¼ to ½ in. thick) possess remarkable stability and wearing qualities. By the same token, materials such as granite have proved themselves excellent aggregates in bituminous mixtures. Water-worn sands, on the other hand, are the most abundant and generally available fine aggregate materials in use, and they are capable of producing good results but, as is quite clear, they require more careful investigation and manipulation than either of the other two classes mentioned in order to insure the best results. Bituminous mixtures of both the sheet and concrete types showing shearing strength of 50 lb. per sq. in. and better, have repeatedly demonstrated their ability to resist distortion under most severe traffic conditions, while otherwise excellent mixtures of 25 to 45 lb. per sq. in. shearing strength have developed ruts and waves (shoves) under the same traffic. Thus it will be seen that mixtures must be designed to withstand traffic of a specific nature and volume; and hence the extreme importance of stability.

Absorption.—Absorption is obviously a function of denseness of mixture and consequently is directly related to compressibility of the mixture.

In general, the limestone mixtures are superior to those employing silica, the Ottawa sand fine aggregate being the only exception. This is conclusive proof that the nature of the sand is the controlling factor in that particular case.

By increasing the amount of bitumen in the silica mixtures, absorption was materially decreased, along with decreased mixture voids and stability.

This test for absorption shows the importance of good compressibility.

SPECIAL FILLERS

Fillers most commonly used in bituminous mixtures include such materials as limestone dust, silica dust, portland cement, both raw and finished, slate dust, etc. Of these, limestone dust is the most extensively used as it is more widely distributed than any of the others. Materials such as asbestos dust and diatomaceous earth (of the Celite type) have also been used with much success. As an example of remarkable properties that may reside in filler materials, the case of diatomaceous silica is cited, since it has been extensively investigated and has demonstrated its merit as a filler.

The material is extremely fine and weighs only about 20 lb. per cu. ft. as against 75 to 95 lb. per cu. ft. for the average filler material. A considerable percentage of its particles are spicules and although its void-reducing properties are slight compared with limestone or silica dust, it imparts remarkable stability to the mixture, and when employed in judicious amounts, increases both compressibility and workability. In one case of its use, it was found that 5 per cent of Celite and 15 per cent of limestone dust produced 2.5 times the stability as in the case of the same aggregate with 25 per cent of the limestone dust. The remarkable stabilizing value of this particular material resides in the nature and shape of its particles.

From the foregoing it will be evident that the field of fine aggregates for use in bituminous paving mixtures is by no means a closed one, either with respect to fillers or sands, natural or manufactured. The last word on suitable aggregates has not yet been written. By the same token, none of the materials should be used blindly but only with a complete understanding of its properties.

EFFECT OF AGGREGATES UPON STABILITY OF BITUMINOUS MIXTURES

By Prévost Hubbard[1]

Synopsis

Stability is defined as resistance to deformation or internal displacement under such stresses as are imposed by traffic.

Mineral aggregates may possess a considerable degree of inherent stability irrespective of the presence of bituminous cement. As they constitute the great bulk and weight of a bituminous mixture it follows that their inherent stability becomes an important part of the stability of the mixture when it is properly designed. Thorough compaction is the primary factor affecting stability, and density of the compacted mass is of next importance. Softness of the mineral fragments may or may not be advantageous from the standpoint of stability depending upon the degree of softness and size of particles.

Various classes of coarse and fine aggregates are discussed with reference to stability, and the fact is developed that the value of mechanical analysis or grading with relation to density is commonly over estimated.

Stability tests on fine aggregate bituminous mixtures have developed considerable information relative to the effect of aggregates upon stability. In general, the addition of mineral filler to any sand will increase stability so long as it reduces voids in the aggregate. When further addition of filler begins to increase voids, then stability of the mixture also begins to decrease.

Two sands of the same grading may possess entirely different stability values, probably due to difference in surface texture. Different mineral fillers may also vary greatly in their stabilizing values due to differences in fineness and surface texture.

There are so many factors affecting the stability of bituminous mixtures that it is impossible to prophesy, by reference to the usual tests to which the individual constituents are subjected, what stability may be produced in mixtures in which they are combined in definite proportions.

Definitions of Terms

In this discussion the term "bituminous mixtures" will apply to the various types of paving mixtures produced by thoroughly and uniformly mixing mineral aggregates with bituminous cements. "Stability" is here defined as resistance to deformation or internal displacement under such stresses as are imposed upon a pavement by traffic.

Bituminous cements are plastics and bituminous paving mixtures are, therefore, "plastic mixtures" in which the degree of plasticity is dependent upon a number of factors, including the character and percentage of the bituminous cement and the temperature of the mixture. In so far as possible those factors relating to the character, quantity and temperature of the bituminous cement will be eliminated from consideration in an endeavor to deal primarily with "effect of aggregates upon stability."

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[1] **Chemical Engineer, The Asphalt Assn., New York City.**

INHERENT STABILITY

Mineral aggregates in themselves may or may not possess a considerable degree of stability or resistance to displacement under traffic. When a mineral aggregate is properly combined with bituminous cement and thoroughly compacted, any inherent stability of the mineral aggregate should become a part of the stability of the mixture. As the mineral aggregate constitutes the great bulk and weight of the mixture it is evident that its own inherent stability must play a very important part in the stability of the finished mixture. In fact, it would seem reasonable to assume that, with other factors under suitable control, the stability of the mixture is a direct function of the stability of the mineral aggregate. Before discussing the relationship of mineral aggregates to the mixture it would seem advisable to classify certain known characteristics of aggregates with reference to their "inherent stability."

Angularity and Compaction:

Starting first with such coarse aggregates as broken stone, crushed gravel, crushed slag and similar products of the size commonly used in macadam wearing courses, we have first to consider the mechanical interlocking of the individual fragments, made possible by their angularity. A pile or layer of such aggregate which has not been interlocked by compaction has little or no stability. Compaction, or closeness of packing, therefore, is the primary factor affecting stability. If coarse angular fragments of approximately the same size are forced together as closely as possible, a certain degree of stability is produced which, however, is insufficient to withstand severe traffic forces tending to produce displacement. Bonding such uniformly sized coarse praticles with a bituminous cement may greatly increase the stability of the aggregate, but experience in highway construction has indicated that in general the presence of fine aggregate is essential, either in the interstices between the large fragments or as a cover for the coarse aggregate.

Density of Compacted Aggregate:

If round gravel pebbles are substituted for the coarse angular fragments previously considered, it will be found that no matter how thorough the compaction, the mass will possess little or no inherent stability because no interlocking of the round pebbles is possible. It is known, however, in the construction of gravel roads, that when the interstices between these pebbles are thoroughly packed with a fine aggregate consisting of smaller pebbles, sand and clay, a high degree of stability may be produced in the mass. Even in the case of angular fragments, inherent stability is greatly increased by the presence of finer particles in the interstices, so the second principal factor affecting stability might be considered as density or freedom from voids of the compacted aggregate.

Hardness:

In connection with density, the physical characteristic of the mineral praticles, known as hardness, may play an important part. In macadam construction a hard, tough rock is usually preferred to a softer rock, not so much, however, from the standpoint of stability of the finished pavement as to its greater ability to withstand the crushing and abrading action of traffic. Provided the proper amount and character of bituminous cement is incorporated with a relatively soft mineral aggregate, little or no abrasion will result from traffic in the finished pavement, and hardness is not, therefore, such an important characteristic. As a matter of fact a relatively soft rock may produce a pavement of greater inherent stability than one which is much harder because of the fact that the corners and edges of the angular fragments break down under compaction, to produce more intimate contact of surfaces and, at the same time, the finer portions so produced help to fill the voids between the coarser fragments. As examples may be cited such products as the soft limerock of Florida and certain soft blast-furnace slags, which, under compaction, break down to produce an almost solid mass with very low void content.

The principal objection to the use of very soft coarse aggregate in bituminous paving mixtures lies in the fact that if so soft that traffic breaks or crushes the individual fragments to an appreciable extent sufficient bituminous cement is not present in the mixture to coat and bind the additional surfaces which are thus produced, and the advantage which should be derived from the presence of the bituminous cement is, therefore, greatly decreased, particularly at the surface of the pavement which is apt to pit and ravel under traffic. As will be seen later, softness is not an undesirable characteristic if the fragments of the aggregate are so small that they are unlikely to be crushed by traffic loads and, for the very finest portion of the mineral aggregate, softness may even prove to be a very desirable characteristic.

Fine Aggregates:

Coming now to fine aggreages, of which the most widely used is sand, a consideration of the first mentioned property of coarse aggregates: namely, angularity, will be found to be of relatively less importance. While interlocking of angular sand grains under compaction actually occurs and to some extent adds to the stability of a fine aggregate, it is well known that the inherent stability of very sharp sand when dry is relatively low as compared with the inherent stability of a compacted angular coarse aggregate. It is evident, therefore, if the same general laws apply to fine aggregates as to coarse aggregates, that density or freedom from voids of the compacted mass becomes of increasing importance. From personal observation, the author considers that for use in bituminous mixtures hardness of the individual fragment is not nearly so important a characteristic in sands as in coarse aggregates and that a certain degree of friability may even be advantageous both from the standpoint of increasing density under compaction and facilitating compaction under methods commonly employed in highway work. By this it should not be inferred that very soft sand grains are preferable to moderately hard grains, but the property which same sands possess of losing their sharpest corners and angles under compression seems to produce increased stability in the mixture. Stone or slag screenings may be as satisfactory as sand, but the difficulty in securing such products with a range in mechanical sizes which will produce dense mixtures under compaction in general handicaps them in comparison with sands.

Mineral Fillers:

In connection with securing a desirable degree of density in fine aggregates for bituminous paving mixtures, experience over a long period of time has demonstrated the desirability, if not the absolute necessity, of incorporating with the sand an extremely fine aggregate known as mineral filler. The same is true of coarse aggregate bituminous paving mixtures in which, however, the presence of filler is of perhaps less importance. Most natural sands and fine aggregates incidentally produced by crushing mineral matter in the manufacture of the usual commercial sizes of coarse aggregate do not contain a high percentage of extremely fine particles which might be designated as impalpable powder, and in order to secure such products artificially fine grinding or pulverizing becomes essential. Limestone dust has been more widely used than any other mineral filler for bituminous mixtures, and it has been found that the presence of this dust in suitable amount not only increases the inherent stability of the mineral aggregate, but probably raises the efficiency of the bituminous cement as a stabilizing agent by imparting a characteristic to the mixture commonly known as toughness. Other mineral dusts or impalpable particles such as slate, shale, oyster shell, silica, diatomaceous earth, portland cement, hydrated lime, and even clay, have been successfully used as substitutes for limestone filler and, from investigations which have been conducted, it appears quite probable that many other products may prove satisfactory in so far as their stabilizing effect is concerned. While not conclusively demonstrated by reliable laboratory or service tests, indications point to the fact that among such products those which are soft, and perhaps somewhat porous in character, are more effective than the harder and denser fillers, not so much in their void filling characteristics as in the effect they produce upon the bituminous cement.

Function of Bituminous Cement:

The function of the bituminous cement in a bituminous mixture is to waterproof the structure and to bond the individual particles of mineral aggregate together to such an extent that the inherent stability of the aggregate is increased to the degree necessary to prevent its displacement under traffic. This function should be performed without losing all characteristics of a plastic mixture, as the life and serviceability of the mixture is to a considerable extent dependent upon its plasticity. Frequently a high degree of stability in bituminous paving mixtures may be obtained by using a much lower percentage of bituminous cement than experience has demonstrated to be advisable. When this is done the mixture at low temperatures may possess most of the undesirable characteristics of a rigid structure. Film thickness of bituminous cement seems to be an important factor affecting both stability and life of a bituminous mixture and this factor is controlled by both the character and the degree of density of the compacted aggregate. The term "film thickness" as here used does not apply merely to the thickness of film of bituminous cement which would surround an individual particle of the mineral aggregate but rather to the entire bituminous content of the mixture. Irrespective of density after compaction some mineral aggregates appear able to carry a greater film thickness of bituminous cement than other aggregates without decrease in stability, and this characteristic is believed to be highly desirable.

STABILITY TESTS

Although considerable work has been done in recent years in developing a test for determining the stability of coarse aggregate bituminous mixtures, the development of such tests for the fine aggregate type of mixtures has advanced more rapidly and, from studies of the type known as sheet asphalt mixture, a considerable amount of information has been developed relative to the effect of aggregates upon the stability of bituminous mixtures.[1] Without discussing individual investigations that have been conducted, a review of the present available data would seem to indicate the following facts in connection with mixtures of sand, mineral filler and bituminous cements. From the preceding discussion it would seem that the same facts should hold true for the coarse aggregate type of mixtures.

Where a bituminous mixture is composed of an admixture of two or more mineral aggregates in combination with a bituminous cement, evaluation of the individual aggregates with reference to the stability that may be secured in the mixture, can be only roughly approximated. In other words, the proportions in which the aggregates are combined is such an important consideration that it is necessary to take into account the effect of variations in proportions. Thus while two sands may be approximately evaluated from the standpoint of stability, the stability produced in a bituminous mixture with either of these sands will be very greatly influenced by the character and proportion of mineral filler incorporated in the mix.

Starting with a given sand and properly controlling the percentage of bitumen the stability of mixtures produced with this sand is increased to a certain maximum by the addition of increasing amounts of mineral filler, after which the further addition of filler produces a decrease in stability. The percentage of filler producing maximum stability with a given sand seems to closely approximate the point at which the lowest percentage of voids is produced in the compacted mineral aggregate. Most mineral fillers, while exceedingly fine, contain a relatively high percentage of voids when compacted and, if used in excess, will, therefore, increase the percentage of voids in the total mineral aggregate beyond the minimum possible percentage obtainable by using the proper proportions.

When combining the same mineral filler with two sands individually to produce mixtures as nearly identical as possible the stability of these mixtures may be entirely different. This has been found to be true between two sands from different sources which have been brought to the same identical grading as determined with laboratory testing sieves. It has been found that when two such sands are combined with the same percentage of mineral filler the percentage of voids in the compacted aggregates may differ materially and that

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[1] Prévost Hubbard and F. C. Field, "A Practical Method for Determining the Relative Stability of Fine-Aggregate Asphalt Paving Mixtures," *Prpceedings*, Am. Soc. Testing Mats., Vol. 25, Part II, p. 335 (1925).

that aggregate containing the lowest percentage of voids is not necessarily the one producing greatest stability. Some characteristic of the sand, other than its mechanical grading, is, therefore, responsible for this difference.

While the results of some investigators have indicated that in such cases greater angularity of sand grain may cause the greater degree of stability, investigations by the author have indicated that some other factor, which, for want of a better name, has been termed "surface texture" may be responsible. Thus, between two sands of the same grading in which no visible differentiation under the microscope could be obtained, the sand showing the higher adsorption of asphalt, when treated with a solution of asphalt dissolved in carbon tetrachloride, was the one which produced mixtures of greater stability. It has been found that while the relative stabilizing values of two sands may perhaps be indicated by their difference in adsorption value, difference in adsorption itself is not probably responsible, because when the two sands were mixed with a lubricant the same relative difference in stabilizing value was noted. Just what characteristic is responsible for such differences in stabilizing values remains to be ascertained, but the mechanical grading and visible characteristics of the sand as determined by ordinary methods will not properly evaluate them.

Considerable differences in stabilizing value of different mineral fillers, even of the same type, have been noted when the fillers have been combined with two or more different sands. While these differences are in part probably due to differences in degree of fineness, this characteristic in itself does not explain the matter satisfactorily. "Surface texture" may here also play an important part. Although one of the functions of a mineral filler is to fill small voids in the coarse mineral aggregate, it is, of course, impossible to so distribute it that it does nothing but fill voids. As a matter of fact, when used in customary proportions, it actually serves to separate the sand grains to some extent, at the same time reducing the percentage of voids in the total aggregate. The degree of separation of sand grains which is produced will depend upon the percentage of filler used and its relative coarseness or fineness.

EVALUATION OF BITUMINOUS MIXTURES

From the above it is readily seen that a bituminous mixture is by no means a simple proposition and that the inter-relationship of the various factors affecting stability of the mixture is exceedingly complex. With a given sand aggregate, mineral filler, and bituminous cement it is impossible to prophesy, by reference to the usual tests to which such individual constituents are subjected, what stability may be produced in mixtures in which they are combined in definite proportions. This can only be learned by making actual stability tests upon such mixtures.

The problem is further complicated by the fact that stability of a properly proportioned mixture is almost a direct function of its degree of compression or compaction and, from a practical standpoint, workability and ease of compression are most important factors to be considered in designing paving mixtures. Methods of compressing mixtures in the laboratory and on the street are quite different and it is not always easy to duplicate on the street the degree of compression which is secured by laboratory methods. It has been found that some mineral aggregates will produce highly stable mixtures in the laboratory which degree of stability cannot be duplicated on the street because of difficulty in securing the same degree of compaction by construction methods. It is evident, therefore, that in such cases the relative stabilizing values of the aggregates should not be evaluated solely on laboratory tests made on specimens prepared by only one method of compaction. Where laboratory methods of compaction are varied within certain limits and the stability of specimens so produced from the same mixture are found to lie close together, good workability and ready compression are indicated from the standpoint of street operation.

It seems probable that too much importance has been assigned to the screen or sieve analysis of mineral aggregates to be used in bituminous paving mixtures. While the importance of density of the compacted aggregate is unquestioned it is the author's opinion, substantiated by considerable research, that the density of the compacted aggregate cannot safely be estimated from the screen or sieve analysis of the mineral aggregate, particularly when such sieve or screen analysis is made upon individual aggregate constituents which are to be combined in the total mineral aggregate. It would seem that the principal value of the screen or sieve analysis lies in ascertaining that a reasonable distribution of the various size particles exists and that no particular significance relative to density or stability may be attached to exact limitations in percentage of any individual size or sizes which may be present. Reasonable distribution of the individual sizes is known to be desirable from the standpoint of prevention of segregation preceding and during the operations of mixing and laying, and should be primarily considered from this standpoint.

INFLUENCE OF QUALITY OF COARSE AGGREGATE ON STRENGTH OF CONCRETE

BY F. C. LANG[1]

SYNOPSIS

The tests and characteristics by which the quality of coarse aggregate is ordinarily judged are enumerated. The effect of each of these properties on the compressive, flexural and tensile strength of concrete so far as could be found in existing published research data is shown. This information is taken from investigations which were primarily made for other purposes. No comprehensive published research data pertaining to this particular subject was found.

Conclusions drawn from the data available are:

1. Coarse aggregate deficient in structural strength as shown by the abrasion test has more effect on the flexural and tensile strength of concrete than on the compressive strength.
2. The absorption in itself has no effect on the strength of the concrete when the effective water-cement ratio is correctly used.
3. There are not enough data to warrant any conclusions as to the effect of shape and surface texture.

In this paper coarse aggregate is considered as that part of the concrete mix which is retained on a $\frac{1}{4}$-in. screen, and may be either crushed stone, gravel or blast-furnace slag. The quality of the coarse aggregate is ordinarily judged by:

1. Resistance to wear (Deval abrasion test);
2. Toughness and hardness;
3. Absorption;
4. Properties which are difficult to gage, such as shape and surface texture.

The effect of soundness or durability of the aggregate upon the strength of the concrete is not considered in this paper. There can be no question but that the use of unsound aggregates will eventually affect the strength of the concrete. This subject is, however, covered elsewhere in this symposium.[2]

Cleanness and grading of aggregate are largely a matter of plant operation rather than an inherent quality of the material and for that reason are not discussed here.

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[1] Engineer of Tests and Inspection, Minnesota Highway Department, St. Paul, Minn.

[2] F. R. McMillan and G. W. Ward, "Influence of Coarse Aggregate on the Durability of Concrete," see p. 816.

The strength of concrete is measured by the compression, flexure and tension tests. The available published information which shows a definite relationship between the quality tests of aggregates and the resulting strength of concrete is meager. In order to draw the conclusions which are given in this paper, it has been necessary to refer to published investigations in which the kind of stone was given even though the regular quality tests on the aggregate were not. A point which must be given consideration in examining the published data is that in some of the investigations the water-cement ratio was maintained the same for all aggregates while in others, especially the earlier investigations, the aim was to maintain a uniform consistency.

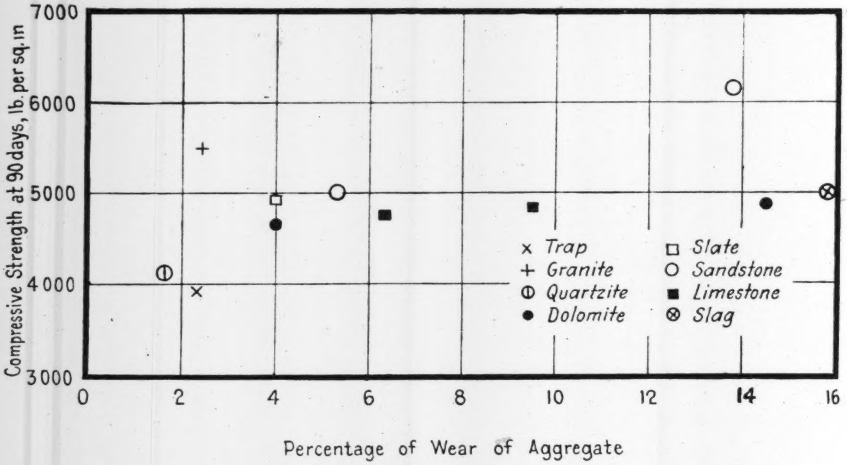

FIG. 1.—Relation Between Abrasion Values on Aggregate and Compressive Strength of Concrete.

(From data by Jackson and Pauls (**4**)[1])

RESISTANCE TO WEAR

Compressive Strength of Concrete.—For a considerable range in abrasion loss of stone there is no corresponding variation in the compressive strength of the concrete. This holds true on all investigations studied (**1**, **3**, **4**, **5**, **6**, **7**, **9**).[1] Figure 1, plotted from data taken from "Accelerated Wear Tests of Concrete Pavements" by F. H. Jackson and J. T. Pauls (**4**), is typical. There is, of course, a limit, for very high abrasion loss will show a stone of such low structural strength that the compressive strength of the concrete would be affected. A sandstone having an abrasion loss of 11 to 13 per cent is given in six of these reports. This in all cases gives as high compressive strength concrete as does the granite or trap (1, 3, 4, 6, 8, 9). Just how much higher the abrasion loss could be without affecting the strength of concrete, available information does not show. Obviously the richness of the mix would be a factor. The Deval abrasion loss does not necessarily show the structural strength of the aggregates. Granular material like sandstone has a higher abrasion loss for the same structural strength than does a material such as limestone.

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[1] The boldface numbers in parentheses refer to the papers given in the list of references appended hereto.

Flexural Strength of Concrete.—The flexure test will show decreased strengths due to low resistance to abrasion of the aggregate before there is any effect on the compressive strength (6, 9, 11, 12). The same sandstone previously referred to shows a considerably lower flexural strength of concrete. This is not apparent on the lean mortars or in rich mortars tested at early ages. Gravels having a high percentage of wear due to the presence of some very soft particles show a low flexural strength concrete (12). The value of the abrasion test on coarse aggregate depends on its indicating the aggregate with low structural strength.

Tension.—Only one investigation (6) showing the tension test of concrete for various aggregates has come to the attention of the author. It is interesting, however, to note that this also shows low strength for the same sandstone referred to in the other tests.

TOUGHNESS AND HARDNESS

For toughness and hardness there are not enough data available to warrant any general conclusions (4, 9). It would appear, however, that these tests are of value only as they show low structural strength of the aggregate.

ABSORPTION

Except as the absorption of the stone may have decreased the effective water-cement ratio in some investigations and thus produced a stronger mortar showing a high-strength concrete both in compression and flexure tests, there is nothing to indicate that there is a relation between the absorption of aggregate and the strength of concrete (1, 4, 6, 8, 10). Stones with low structural strength have a tendency to show high percentages of absorption, and it is only as the structural strength is indicated by the absorption test that it is of any value in showing the strength of the concrete. Some engineers have been of the opinion that rock with high absorption had a better bond with the mortar. One theory was that with rocks having little or no absorption, a water film surrounded the particle, whereas with the more absorptive rocks the moisture was sucked into the stone resulting in stronger bond between the coarse aggregate and mortar. In an attempt to prove or disprove this theory, compression and flexure tests on concrete in which the only variable was the condition of the coarse aggregate were made at the Minnesota Highway Department laboratory. The results are shown in Table I.

The wet stone had been immersed in water for 24 hours, surface dried, and then incorporated in the mix. The dry stone was air dried over steam coils for several months in the laboratory. The absorption on sandstone was 4.58 per cent and for the limestone 1.55 per cent.

TABLE I.—CONCRETE TESTS USING WET AND DRY AGGREGATE.

COMPRESSIVE STRENGTH, LB. PER SQ. IN.

	SANDSTONE			LIMESTONE	
	WET	DRY		WET	DRY
	4545	4675		3801	3288
	4780	4415		4401	3955
	5515	4169		3872	3810
	4394	4535		3799	4235
Average	4809	4449	Average	3968	3822

MODULUS OF RUPTURE, LB. PER SQ. IN.

	SANDSTONE			LIMESTONE	
	WET	DRY		WET	DRY
	697	816		913	949
	731	705		834	840
	705	708		837	824
	749	699		795	867
	738	691		839	831
	706	696		781	852
Average	721	719	Average	833	860

The amount of water used was 5.2 gallons per sack of cement plus the amount absorbed.

The results do not show any advantage for the dry aggregate. The theory has probably been the result of the absorptive aggregates showing higher strengths in tests where the effective water-cement ratio was not the same.

SHAPE AND SURFACE TEXTURE

The compressive strength of concrete is not appreciably affected by the shape and surface texture of aggregates ordinarily used. Some investigations indicate that smooth round particles do not produce as high flexural strength concrete as angular rough particles (2, 9, 11, 12). The question of relative workability must, however, be given consideration. The advantages of the rough angular material might be partially or entirely offset by the necessity of increasing the water-cement ratio or decreasing the amount of coarse aggregate used in order to obtain necessary workability. Enough data are not available at this time on which to base any definite conclusions.

No attempt has been made in this paper to differentiate between crushed stone, gravel pebbles and slag. Any of these materials have widely different characteristics and the superiority of one type over another would depend largely upon the available materials in a given locality. Cleanness, structural strength, durability and grading appear to be the paramount considerations.

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(4) F. H. Jackson and J. T. Pauls, "Accelerated Wear Tests of Concrete Pavements," *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, Table II, p. 870, Table IV, p. 875 (1924).

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(6) H. F. Gonnerman and E. C. Shuman, "Compression, Flexure and Tension Tests of Plain Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, Table VI, p. 544 (1928).

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(10) H. J. Gilkey, "Coarse Aggregate in Concrete as a Field for Needed Research," *Proceedings*, Am. Concrete Inst., Vol. XXIII, Table 2, p. 387, Table 5, p. 393 (1927).

(11) State of Washington Highway Department Laboratory Test for Quality of Coarse Aggregate, not published.

(12) A. T. Goldbeck, "Bound-Brook Tests," Nat. Crushed Stone Assn., *Bulletin No. 5*, Table I, p. 49 (1928).

INFLUENCE OF COARSE AGGREGATE ON THE DURABILITY OF CONCRETE

By F. R. McMillan[1] and George W. Ward[2]

Synopsis

A brief review is presented of the literature bearing on failure of aggregate of various types. Igneous, sedimentary and certain of the metamorphic rocks are discussed.

Consideration is given to texture, structure and those impurities that may cause rock failure. Points to be guarded against in selection of aggregate are pointed out.

The importance of having aggregates of stable mineral composition under the conditions of use, if the concrete is to be of long life, is quite obvious. Unfortunately, however, it is not easy to classify aggregates into suitable and unsuitable groups with reference to this important requirement, either directly from a study of the rock itself or through experience with its use in concrete. The difficulty from rock study lies in the fact that there are so many possible variations in the aggregates of various minerals that unvarying performance cannot always be predicted. The principal reason why experience has not provided definite classification is that differences in concrete due to other factors have largely obscured differences due to aggregates themselves.

Aggregates from many sources have been used successfully for years and in relatively few cases have failures actually been traced to the coarse aggregate. This has created a feeling of confidence in all aggregates that, in some cases at least, borders on one of indifference. Fortunately, confidence can be had in most of the aggregates from established sources, but observation has shown and the literature bears record that not all aggregates are suitable for concrete when exposed to the elements.

The authors have recently begun a serious study of aggregates as they affect the durability of concrete. While it is too early to contribute much to this symposium from this study, they are glad to set forth a brief review of the literature bearing on failure of aggregate

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[2] Petrographer, Portland Cement Assn., Chicago, Ill.

of various types. In this review, which is largely the work of Mr. Ward, effort has been made to present in compact form the essential characteristics of the rock groups which are likely to be encountered in aggregate materials, and to point out to which of these characteristics failure can be attributed.

In presenting this discussion, the aim has been to avoid the use of terms comprehensible to the geologist only, but in the interest of accuracy a certain technical treatment is necessary. If the engineer is to make use of the technical information of the geologist he must learn something of his language. In the discussion leading to this joint effort, it developed that there is need for a more carefully prepared terminology where different groups attempt to bring their combined knowledge to the solution of a common problem.

IGNEOUS ROCKS

Experience has taught that the most durable concrete aggregates are the unaltered igneous rocks. Among these, the dense or aphanitic varieties, as represented by the felsites, are the most resistant. The coarse-grained intrusive rocks, or phanerites, are harder, but due to their larger crystals break more readily under impact. These are the common granites, pegmatites or giant granites, and the gabbros. Rocks may be acid (high in silica) as rhyolites, andesites, and granites, or basic (low in silica) as the basalts and gabbros. It is known among geologists that the more basic or less siliceous a rock is, the lower in general its durability and the easier it weathers.

Among igneous rocks making durable aggregate are trap rocks. The term trap rock, as used by the field geologist, is rather loosely applied to a number of dark, fine-grained igneous rocks as found at the Hudson Palisades. In the literature[1] of concrete, the word trap is still more loosely applied, a hard limestone sometimes being referred to as trap. There are other examples of terms loosely used, such as chert, which add to the confusion in comparing tests made upon aggregates by different investigators.

So far, the literature has still to trace a concrete failure to the use of sound igneous rocks. In general, trap rocks and granites make very desirable concrete aggregates. By some, these are considered to be superior to the best limestone.[2]

Feldspars:

Feldspars are minerals of igneous origin and are far more important as rock constituents than any other group of minerals. They constitute nearly 60 per cent of igneous rocks and serve as the basis of their classification.[1] Most of the trouble with igneous rocks is due to weathering of feldspars. Other major constituents that weather, such as the hornblendes and the pyroxenes, give structures more durable than feldspars because of a better interlocking of the secondary minerals.

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[1] Report, National Conference on Concrete Road Building; *Engineering News*, Vol. 71, p. 433, Feb. 19, 1914.

[2] Engineering Research Division, University of Texas; *Concrete Products*, Vol. 59, August, 1922.

The feldspars form a continuous series from orthoclase, the potash variety, down through the plagioclases. The plagioclases vary in composition from albite, the soda-alumina silicate, through soda-lime varieties to anorthite, the lime member. It has been found that descending in the series, the percentage of lime increases and the durability decreases.[2] Thus, anorthite is more readily weathered than albite or any other member higher up in the series. The potash varieties are more stable. These develop kaolinite under weathering conditions and as the process of hydration goes still farther, it may result in the formation of halloysite, montmorillonite, or other hydrous aluminum silicates.[3] This is believed to be the ordinary method of origin of common clay, although concentration in clay beds is accomplished in some cases by water sorting during transportation.

J. C. Pearson and G. F. Loughlin[4] report an interesting case of dangerous aggregate of igneous origin. The concrete examined had been made with plagioclase (labradorite) as the aggregate. Labradorite is one of the soda-lime feldspars and is near the end of the series. At the close of about one and one-half years the structures were declared failures. Investigation revealed that the mortar had sufficient strength and that the responsibility for failure lay with the aggregate. Petrographic examination showed that the feldspar had undergone considerable alteration before its use as aggregate, when it was still in the form of the igneous rock. This alteration produced such secondary minerals as laumonite, kaolinite, calcite, and zeolites. These, when present in quantity (as they appear to have been here) are particularly detrimental, since the feldspar attains a chalky texture and disintegrates.

SEDIMENTARY ROCKS

Limestones:

Limestones are defined as bedded deposits of calcium carbonate. There are all varieties of limestones and they take their qualifying adjective from the predominating impurity as, argillaceous limestone, cherty limestone, etc. They even grade from magnesium limestones into dolomites which have molecularly equal parts of magnesium carbonate and calcium carbonate as $CaMg\ (CO_3)_2$.

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[1] A. N. Winchell, "Elements of Optical Mineralogy; Part II," p. 292.

[2] F. W. Clarke "Data of Geochemistry," *Bulletin 695*, U. S. Geological Survey.

[3] A. N. Winchell, "Elements of Optical Mineralogy; Part II," p. 294.

[4] *Proceedings*, Am. Concrete Inst., Vol. 19, p. 142 (1923).

Limestones that break with a smooth conchoidal fracture, particularly if the porosity is low, are usually good aggregates. Uneven fractures that tend to follow planes, the presence of many closely-spaced bedding lamellae and high insoluble matter, especially if clay minerals can be microscopically detected, are danger signs. If a high porosity accompanies any of the above the aggregate should be given close scrutiny. Sulfides are objectionable as they oxidize to give sulfuric acid which attacks any calcareous aggregate or lime in the cement, forming gypsum. This gypsum tends to expand in crystallizing, disrupting the concrete and later by leaching out further weakens the mass.

G. F. Loughlin[1] has examined a group of argillaceous limestones. He found that the samples were unsound because clay minerals had concentrated into definite areas. These clay minerals swell to a marked degree when in contact with water and this may bring about disruption. Specimens of limestones were reported upon from Illinois, Iowa, Kansas, and Pennsylvania. Mr. Loughlin has placed the reason for failure of other limestones upon poor binding material. One sample[2] in particular was structurally weak because the original chalky cementing material had not all crystallized into distinct grains of strong calcite, continuous with the calcite of the fossil, but had remained as the original weak chalky calcite of poor binding power. Impure crystalline limestones that contain micaceous streaks and those containing shaly or clayey streaks are not be be recommended as aggregate. Weathering proceeds rapidly along the streaks and the rock may crush into flat scales which are undesirable. Furthermore, while in the concrete, these clay or mica seams might swell by water absorption, and endanger the durability of the concrete.

Dolomite, high-magnesium limestones and marbles are much more slowly weathered than high-calcium limestones and marbles. However, some danger points, such as bedding planes, concentrations of clay minerals, mica seams, sulfides, etc., may occur and one should be on the watch for them.

Sandstones and Conglomerates:

Sandstones are sedimentary rocks composed of grains of sand cemented together. The cementing material may be any of a number of binding mediums among which are silica, calcium carbonate and iron oxide. Numerous sandstones owe their tenacity to the pressure to which they were subjected during or after their time of consolidation. Sandstones are for the most part made up of quartz, but many other durable minerals may occur as feldspar, mica, garnet, magnetite, etc. The finer-grained sandstones grade off into shales and the coarser-grained into conglomerates. Thus, a conglomerate is a cemented gravel, and deleterious substances in gravel would also be detrimental in conglomerate. At least one serious failure has been reported by Cloyd M. Chapman[1] to be due to a conglomerate. Pottsville conglomerate of Carboniferous age, consisting of fragments of granite and syenite cemented with silicia, was used as a fine aggregate in a floor topping. The topping cracked in large rectangles that curled up along the edges of the cracks to produce saucer-like depressions. Mr. Chapman, after many tests made with Pottsville conglomerate, arrived at the conclusion that excessive shrinkage with such aggregate was the cause of failure.

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[1] G. F. Loughlin, "Usefulness of Petrology in the Selection of Limestone," ***Rock Products***, March 17, 1928.

[2] G. F. Loughlin, *Proceedings*, Am. Concrete Inst., Vol. 23, p. 337 (1927).

Flint and Chert:

Flint and chert are frequently confused in the literature. Mineralogists consider chert as an impure variety of flint. Flint is a compact aggregate of cryptocrystalline and hydrous silica. It has a conchoidal fracture and is translucent in thin pieces. Chert, or hornstone, is an impure variety of flint. It is amorphous, but sometimes appears almost crystalline. The fracture is sub-conchoidal and thin flakes are opaque. The origin of these siliceous aggregates is shrouded in doubt. Deposition around a nucleus, such as a sponge spicule, a shell, some organic or inorganic form, has been one suggested method. Replacement of an earlier formation, or colloidal precipitations, are considered to be other methods of origin. At any rate, it seems apparent that much water was present in the material previous to metamorphism. As temperature and pressure act, water is lost and consolidation takes place. This is accompanied by volume shrinkage which produces microscopic cracks. Deposition of minerals in these cracks can bring healing that may or may not increase the strength of the aggregate, depending upon the size of crystal grain deposited, the bond between these crystals, and the bond between new material and the old. Those with unhealed cracks readily split under temperature changes even to the extent of "popping." The familiar experiment of boyhood days, of placing a piece of chert in a bonfire to hear it explode, will be recalled.

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[1] Cloyd M. Chapman, "Influence of Aggregates upon Shrinkage of Mortar and Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 24. Part II, p. 767 (1924).

"Chats" or finely crushed chert tailings produced in the milling of lead-zinc ores in the Missouri-Oklahoma-Kansas mining region are placed on the market. This material is considered to be of good quality[1] and concrete with this aggregate has been successful. Cases have been cited,[2] however, where "chats" have given unsound concrete. "Pop-outs" are the chief cause of complaint. F. V. Reagel, Engineer of Materials,[3] Missouri State Highway Department, discusses chert from the Burlington Formation as follows:

> The effects as observed on concrete containing chert aggregate are serious both as to the appearance and the strength of the work. Whenever a piece of chert has a position in the concrete near enough to the surface to be readily subjected to the freezing and thawing action of a winter season, the resulting expansion ends in radiating cracks as well as spalling and the pop-outs mentioned above. The final result is a considerable weakening of the concrete in pavement or structure. In case the entire coarse aggregate content of the concrete is the chert mentioned, a sufficient number of cycles of freezing and thawing will result in a virtually complete disintegration of the concrete as rapidly as the progressive disintegration exposes fresh surfaces of the structure to the weathering effects. The presence of small quantities of the chert in the coarse aggregate results in a varying amount of surface effect and strength reduction. The deleterious effect has been verified by both laboratory test and field observation and action has been taken by the Missouri Highway Department limiting the percentage of the objectionable material to 5 per cent of the coarse aggregate by volume.

Samples of this chert obtained by A. A. Anderson of the Portland Cement Association, when subjected to freezing and thawing, completely disintegrated after a few cycles. When subjected to the sodium-sulfate test the material was sound.

The "powdering" of chert is difficult to explain, but according to George P. Merrill,[4] it may be due to the removal of a small percentage of interstitial $CaCO_3$ or may even be a change of physical form. E. C. E. Lord has stated[5] that cherts that do stand up probably do so because of the development of hydrated opaline silica. This silica has a high binding power. He considers that the silica is derived from solution of part of the rock followed by deposition. Thus, partial decomposition may result in a strengthening of the rock.

Herbert F. Kriege, France Stone Co., has carried out investigations[6] at the France Stone Co.'s laboratory upon 100 per cent chert, cherty limestone (50 per cent chert), and chert-free limestone, derived from three different sources. Concrete specimens were made and subjected to freezing and thawing, dry thermal expansion (up to 175° C.) and the sodium-sulfate test. The same series of tests except the dry thermal expansion were carried out on the aggregate alone. Absorption of these aggregates and sodium-sulfate observations on a number of cherts is also reported. A chemical analysis of a 100-lb. chert boulder from the Burlington formation showed good agreement in composition between the chalky exterior, white dense interior, and the translucent center. Unsoundness of aggregates is considered and the last paragraph of Mr. Kriege's article reads:

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[1] *Concrete*, Vol. 12, p. 90, March, 1912.

[2] F. V. Reagel, *Rock Products*, April 16, 1927, p. 90.

[3] F. V. Reagel, *Engineering News-Record*, August, 1924.

[4] G. P. Merrill, "Rock Weathering."

[5] E. C. E. Lord, U. S. Department of Agriculture *Bulletin No. 31*, p. 27.

[6] *Rock Products*, April 27, 1929, p. 50.

> In conclusion, it must be stated that many cherts are not unsound or unstable. The two factors which seem to contribute most to the instability of certain cherts are the microscopic irregularities of texture and the presence of firmly imbedded foreign crystals, such as pyrite, with their different physical properties."

Clay and Its Metamorphosed Equivalents:

A clay is defined as any hydrous aluminum silicate. It is a mixture of fine grains. Clays may be fine or coarse, silty or gravelly, concretionary or fossiliferous, etc. The metamorphism of a clay produces a shale. Further metamorphism results in a slate, and even a fine-grained dense rock termed argillite may be developed if differential stresses are not set up. The cleavage planes in a shale are the original bedding, whereas in a slate stresses produce new planes that are at an angle to the bedding which has ceased to exist as a line of parting. If a shale has been consolidated by pressure alone, there is a great tendency for it to break down upon exposure. Indurate a clay by heat and a shale, which will disintegrate less readily by weathering, results. Frequently, iron sulfides are present in clay and their metamorphosed equivalents. As explained previously these may contribute to disintegration.

If the proper chemical elements are present a schist will be the end product of the metamorphism of clay. In this case, the foliated structure that is characteristic of this rock is produced by deep burial and rock flowage under the influence of heat and pressure.

Concentrations of clay and clay seams that may occur in rocks, chiefly those of calcareous origin, were discussed above under limestones.

Thus, in consolidated clays or shales and in clay slates, there are three factors that may prevent their use as aggregate. One of these factors is the presence of sufficient concentrations of clay minerals which have high swelling power due to absorption of much water. A second is bedding planes, which permit access of weathering agents and thus facilitate decomposition. Specimens of argillaceous limestone examined at the Research Laboratory of the Portland Cement Association separated along the bedding after approximately two hours' water soaking. Apparently, concentrations of clay occurred throughout the specimens, particularly along bedding planes. The third factor is the appearance of sulfides. These are commonly marcasite, pyrite, and pyrrhotite. There are few clay rocks such as the above that do not contain at least one of these sulfides. Pyrrhotite is a little unusual, but the other two are common in occurrence. Marcasite weathers with extreme ease and pyrite does not offer great resistance. The result of weathering is the production of sulfuric acid to give sulfates.

Cleavability of slates is the obvious reason for their exclusion.

The porosity of schists is generally high and the parallel arrangement of the platy minerals gives planes of weakness along which weathering may readily take place. This weathering produces a swelling of the micas and a general softening of the rock.

It is chiefly from rock of the above character that flat particles result. These are objectionable in concrete as they give harsh mixtures and rise to the surface.[1] Their ready disintegration upon exposure to weathering produces pit-holes in concrete which are particularly deleterious in wearing surfaces. In large percentages the softening of these aggregates would seriously affect the strength of concrete. On the other hand, protection offered by a good mortar might be sufficient to prevent serious results.

MISCELLANEOUS

A. F. Coleman[2] reports a badly disintegrated concrete foundation. The mix was 1 part portland cement to 3 parts sand to 6 parts mine rock. The concrete appears to have been thoroughly mixed and properly placed. Other nearby structures for which aggregate from another source was used stood up well, though the atmosphere was highly charged with sulfur fumes. Mr. Coleman gives as the reason for disintegration the formation of sulfides in the concrete by the action of water upon the aggregate. Analysis of the aggregate revealed small percentages of nickel, copper, iron and sulfur. In all probability, what really occurred was the formation of sulfuric acid and the resulting removal of some of the mortar. The mine rock apparently carried sulfides of the metals revealed by analysis.

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[1] J. B. Watson, *Rock Products*, May 30, 1925, Vol. 28, p. 65.

[2] *Engineering Record*, June 1, 1912, Vol. 65, p. 613.

INFLUENCE OF MINERAL COMPOSITION OF AGGREGATES ON FIRE RESISTANCE OF CONCRETE

By S. H. Ingberg[1]

Synopsis

Fires and fire tests have shown a marked difference in fire effects on concrete made with different kinds of aggregates, which have been shown to be caused by differences in mineral composition of the aggregates involved. Four general groups are recognizable as it concerns such effects: (1) calcareous, (2) feldspathic, (3) granites and sandstones, and (4) siliceous aggregates.

The mineral composition of the coarse aggregates has a greater influence on the fire-resistive properties than that of the fine aggregates, due to the greater amount entering into the mix and possibly to the coarser gradation.

For a considerable range in fire conditions in buildings, concrete can be made of any otherwise acceptable aggregates irrespective of mineral composition without danger of collapse of structural members from a fire completely consuming combustible contents and trim. For more severe fire exposures, the fire resistance of concrete subject to severe fire effects can be materially increased by placing metal ties or mesh in the outer portions of the concrete members or structural protections. Further improvement is obtainable by applying plaster or similar protections outside of the structural concrete. Concretes made of aggregates in groups (1) and (2) will generally require no special provisions in their application.

There is need for further work on the effect of the fine aggregate on fire resistance, the fire resistance of concrete made with mixtures of aggregates of different mineral compositions, effect of impurities such as of silica in limestone, and the development of a standard test for fire resistance that will classify concrete with respect to fire resistance without reference to the type of structural member in which it is applied.

The differences in fire effects on concrete produced by similar conditions of fire exposure have been determined as being due mainly to the mineral constituents of the aggregates involved. On the basis of results of fire tests conducted here and abroad, four general groups of aggregates can be discerned as concerns the performance of the concrete made with them.

Group I. Calcareous Aggregates

The group giving the least disruptive effects and the lowest temperature transmission includes calcite and dolomite as contained in high-calcium and calcium-magnesium limestones, sand and pebbles. The temperature travel within the material from fire exposure is retarded by the calcination of the limestone which involves a change from the carbonate to the oxide requiring some 430 calories per gram for calcium carbonate and 340 calories per gram for magnesium carbonate. The calcined material itself possesses good heat-insulating properties. The melting points of the oxides formed are considerably higher than temperatures occurring in building fires even with the impurities ordinarily present, hence effects from fusion should not be expected. After the fire, on exposure to air for a few weeks the oxides hydrate or "air-slake" and fall off, and hence surfaces of concrete made with calcareous aggregates will require repair after fire exposures of any considerable severity.

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[1] Chief, Fire Resistance Section, U. S. Bureau of Standards, Washington, D. C.

GROUP II. FELDSPATHIC AGGREGATES

The second group comprises mainly igneous rocks that as incorporated in concrete do not induce cracking or spalling on the application of heat of the intensity incident to building fires. The predominating minerals present in natural rocks within this group are feldspars, olivine, and amphiboles and pyroxenes such as hornblende and angite. Only very minor amounts of silica in the form of quartz, chert, or flint are present. The more common rocks belonging to the group are basalt, diabase, dolerite, andesite, gabbro, and syenite. The term "trap" is often applied to the dark-colored members of this group but in some parts of the country it is also applied to hard sandstones. Within this group, so far as fire effects are concerned, can also be placed pumice. This is a light volcanic rock of variable composition but in general corresponding to highly siliceous glass such as rhyolite. The temperature transmission from fire exposure through concrete made with it is likely to be no more rapid than through concrete made with calcareous aggregates due to water retained by the cellular structure of the aggregate.

The artificial aggregates that come within this group comprise broken bricks and other burnt-clay aggregates, blast-furnace slag and the harder varieties of furnace clinkers or cinders. The binding media present are largely calcium-aluminum silicates.

The fusion point of concrete made with a number of the aggregates in this group lies within the range 2100 to 2300° F. (1149 to 1260° C.) and fusion of such concrete has occurred in some severe building fires. With the temperatures incident to the standard furnace test, fusion at the surface begins near six hours and considerable fluxing away of concrete takes place after seven hours. However, this action has not greatly decreased the value of concrete protections in tests lasting eight hours.

The temperature transmission from fire exposure through concrete made with aggregates of this group is in general more rapid than for that made with aggregates of the preceding group, although after light or moderate fire exposures less damage will result since the aggregates do not calcine.

GROUP III. GRANITES AND SANDSTONES

The third group comprises igneous rocks higher in free silica than those in the preceding one, or of re-cemented siliceous rocks. The igneous group included are the granites which are composed mainly of somewhat variable amounts of quartz, feldspar and ferromagnesian minerals such as hornblende. Sandstone, quartzite and quartz-bearing schist consist largely of grains of quartz bound with calcareous, ferruginous or siliceous cement. Other minerals often present in various amounts are feldspar, clay, and mica.

On exposure to fire, concrete made with the aggregates in this group is likely to develop cracking, spalling, and similar disruptive effects although not to the extent applying for the minerals in the following group. The bond of some sandstones is disintegrated by the fire exposure leaving them very soft and friable.

From the standpoint of general fire-resistive properties of the concrete concerned, the softer grades of cinders, particularly those with high unburned coal content, can be placed within this group.

The fusion effects for concrete made with aggregates of this group are about the same as given for the preceding group. The temperature transmission is likely to be somewhat higher because of cracking and occasional spalling of the concrete from the fire exposure. Cinder concrete is not subject to such effects, but temperature transmission from fire exposure is aided by the porous nature of the concrete and the combustion of unburned coal where present in large amounts.

GROUP IV. SILICEOUS AGGREGATES

The fourth group comprises rocks consisting mainly of silica not combined to form silicates. Quartz, chert and flint are the principal minerals concerned. The terms "chert" and "flint" are applied to minerals containing micro-crystallin or amorphous forms of silica as distinguished from the visibly crystallin structure obtaining for quartz.

The cracking and disruption of concrete made with aggregates in this group is thought to be caused by abrupt volume changes.

Quartz and also some of the other forms of silica undergo decided volume change near 1063° F. (573° C.), the inversion point from the alpha to the beta form, the change extending over a considerable temperature range if the heating is rapid. Liquid inclusions and water in hydrous forms of silica may also be the cause of some of the fire effects sustained.

Since the most usual fractures are formed at depths of one inch or more from the exposed surface, a stress condition must be set up by the fire exposure that increases with distance from the exposed surface within certain limits, until rupture occurs. Except for occasional disintegration of individual particles the aggregates and concrete outside of the plane of fracture will generally be sound.

The fusion points of concrete made with aggregates in this group are above points that would require consideration from the standpoint of exposure in building fires.

EFFECT OF FORM AND SIZE

In the preceding discussion no distinction has been made between the crushed rock and the pebbles of similar mineral composition since the results of fire tests do not indicate any decided difference from the standpoint of fire resistance of the concrete. There are, however, indications that the effect of mineral composition of the fine aggregate is somewhat less decided than that of the coarse aggregate, even considering the relatively smaller amount present in the mixture. No tests have been made to obtain specific information on this feature.

EFFECT OF IMPURITIES AND MIXTURES

While the tendency in rock formation is toward minerals of a given composition with no large inclusions of impurities, such are occasionally present in amounts that might affect considerably the fire resistance of the concrete. This is particularly true of pebble and sand deposits. The fire tests made with concrete at either end of the fire resistance scale generally apply only for concrete made with calcareous or siliceous aggregates, with not more than 5 to 20 per cent of impurities or minerals not belonging to the classes concerned. There is, therefore, need for tests covering a range in impurities and mixtures for the different classes of aggregates concerned.

EFFECT OF TYPE AND FORM OF STRUCTURAL MEMBERS

The fire tests of structural members and protections of concrete have been mainly of columns and floors. The tests do not show much difference between round and square columns from the standpoint of fire effects. It would be expected that horizontal members such as floors and beams would be more likely to lose protection from cracking and spalling than vertical members such as walls and columns. While the test results and examination of fire ruins do indicate a difference in this direction it is not large. The fire resistance of reinforced concrete floors and columns made and protected with concrete from the aggregates in Group IV as defined above, based on ability to carry working load, is within the range of two to four hours of the furnace test. For a given thickness of protection and concrete made with the aggregates in groups III and IV, a somewhat higher fire resistance will be given by the reinforced concrete members than by the concrete-protected structural-steel or cast-iron members on account of better bond between the protection and structural portion.

There is a field for a fire-resistance test of concrete that will grade concretes made with the different aggregates, at least relatively, without respect to the structural member in which they may be applied. As test specimen, a vertical or horizontal cylinder or prism exposed to fire on all sides, with load applied over all but the area assumed as protection, might serve the purpose.

CONSIDERATIONS GOVERNING PROPER APPLICATION

Since each locality must use the concrete aggregates available within a given shipping radius, these must be utilized in a manner that will secure the needed fire safety for the structures in which they are used. From the standpoint of public regulation of building construction, the building can properly be required to be able to withstand a complete burning-up of combustible contents and trim without collapse of any major structural members, such as columns, floor members and bearing, party and fire-division walls, and that exterior, party and fire division walls provide proper protection against spread of fire. Building owners and fire underwriters are further interested in keeping the damage inflicted within limits that will permit repair without too great expense.

In order that these objects may be attained without excessive or unnecessary cost it is necessary to recognize the difference in fire resistance of concrete made with the different classes of aggregates and to identify the aggregates available in a given locality from the standpoint of mineral composition. The local names of aggregates are not always a reliable index of the minerals actually present. For purposes of public regulation, if the four groups outlined above are deemed too many for convenient enforcement, the aggregates in groups I and II and those in groups III and IV can be combined and the fire resistance of the two respective classes based on performance of concrete made with group II and group IV aggregates, respectively.

With the proportions of building members and thickness of concrete protection generally used, the fire resistance developed will be high enough to resist fires incident to the lighter occupancies from the fire hazard standpoint irrespective of the mineral composition of the aggregates concerned. For greater amounts of combustible contents and building trim, the fire resistance of concrete made with aggregates in groups III and IV can be increased by placing metal ties or mesh near the exposed surfaces. Further improvement can be obtained with plaster or similar protection applied outside of the structural concrete. For floors the lower portion of the slab up to one-half or one inch above the reinforcement can be made with the more fire-resistive aggregates. Concrete made with aggregates in groups I and II will generally require no special provision in their application other than securing proper thickness of protective concrete.

It thus becomes necessary not only to recognize differences in fire resistance of concrete made with the different aggregates but also in the severity of fires to be expected with the various building occupancies or amounts of combustible materials associated with them. Only thus can the best utilization be made of the sources of aggregate supplies available in a given locality.

AGGREGATES FOR LOW-COST UNTREATED ROADS

By C. N. Conner[1]

Synopsis

This paper is based on information collected by the author during an investigation of the low-cost road situation.

Data and standards are not lacking on aggregates for high type roadway surfaces, but correlated information on aggregates and binders for secondary roads is lacking. The object of this paper therefore is to present information on aggregates successfully used in the lower types of surfacing.

This was collected by a study of 43 specifications, 64 patents, 40 annual reports, technical articles and text books. These were supplemented by field inspections and conferences with engineers who were actually and successfully building roads with such aggregates.

The characteristics or points considered are source of supply, preparation, physical characteristics, quality, gradation and serviceability. The use of such admixtures as asphalts, tars and chemical binders are not considered in this paper—except this statement that desirable aggregates are those which may be successfully treated with bituminous materials.

Practically all roadway surfaces contain two essential elements, aggregate and binder. Among the untreated types of surfacing, gravel is most commonly used as aggregate with clay as binder.

The quality of low-cost aggregates varies from soft and friable to hard and durable when local materials are used. Gradation is governed by several factors; two of the most important are ease of maintenance and stability under traffic.

Source of Aggregates

Materials suitable for untreated surfaces or bases are sometimes found in pits at or near the ground surface, along the roadside or within a short-haul distance of it. Such materials are sand-clay and topsoil in the South and Southwest, shale in West Virginia and Pennsylvania, disintegrated granite in the West and Southwest, gravel in many of the states, chert in Georgia and Alabama, caliche in several of the southwestern states and shell on the seaboard. Occasionally the graded road is composed of serviceable material and requires no immediate surfacing. Arizona has stretches of road of this character. Gravel, slag, crushed stone and lime rock are frequently furnished by commercial plants, although it is not unusual for the project contractor or state and county forces to produce their own aggregate from local gravel and stone deposits.

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[1] Engineer Executive, Am. Road Builders' Assn., Washington, D. C.

Such binders as clay and volcanic cinders for untreated surfaces are usually obtained locally and not from commercial plants.

REQUIREMENTS FOR AGGREGATES

When quality is determined by means of tests, such tests are for percentage of wear and toughness, although the toughness test is not as frequently specified as the wear test. Many specifications simply call for "hard, durable" material. Nearly all specifications require that aggregates shall be clean and free from "foreign" material.

The gradation requirements for aggregate are principally affected by the purpose for which they are to be used. If the material is intended for base construction, softer aggregates and larger sizes are sometimes permitted than for wearing courses; but surfaces which are maintained by machining and dragging are usually composed of durable aggregates whose maximum size is less than 1 in.

Sand-Clay.—This mixture for best results should contain sand composed of hard durable grains, preferably angular in shape. Silt is usually present by tolerance. The clay should be sticky and non-slaking in character. The total sand content should be 65 to 80 per cent of the total, silt not more than 15 per cent, and clay not more than 20 per cent. Sand in sizes above No. 10 sieve is particularly desirable.

Chert.—Several kinds of chert may be found in one county according to experience in Alabama; one very hard and durable, another soft and chalky. There are no standard tests for chert, but its suitability and durability are determined by usage. A typical specification in Georgia requires that all material pass a 1½-in. screen and at least 60 per cent be retained on a No. 10 sieve.

Shale.—There are various classes of shale but those which have given best results under traffic in West Virginia contain more than 55 per cent of silica and more than 4 per cent of iron oxide. The maximum size of particle allowed in that state is 3 in. Under traffic and maintenance the larger pieces are broken down to much smaller sizes.

Disintegrated Granite.—The wearing quality of this material is furnished by the silica content which is more than 60 per cent of the total. The bonding properties are furnished by the iron and calcium content, which form over 6 per cent of the total. The material is commonly placed on the road in sizes up to 4 and 6 in. but is broken down in size to less than half under maintenance, traffic or rolling.

Gravel.—Surfacing gravel is usually specified as a "hard and durable" material. Clay in excess of 20 per cent is not usually tolerated. Ten to fifteen per cent total of clay and loam is considered better practice as binder material, but non-slaking binders such as iron oxide and limestone are preferred. In California 50 per cent of the gravel must consist of crushed particles, because of the greater stability of crushed aggregate under traffic. Experience generally has shown that best results are obtained when all the aggregate passes a 1-in. screen, and where a high percentage, 35 or more, is retained on a No. 10 sieve.

Gravel, Slag and Stone for Traffic-Bound Surfaces.—The quality requirements for gravel, slag and stone for traffic-bound surfaces are usually high; that is, the particles must be hard and durable. As specified they are equal to those used for water-bound and penetration macadam, or highest quality gravel roads. Screenings when used are produced from hard rock. In gradation, best results are effected when all material passes a $\frac{7}{8}$ or $\frac{3}{4}$-in. screen. Practice varies as to the lower limits. Ohio practice for the fine material permits about 10 per cent through a No. 10 sieve; Tennessee uses all stone through a $1\frac{1}{2}$-in. screen including dust of fracture.

Stone for Macadam.—For base courses of macadam softer aggregates are tolerated than for wearing courses. For example, in Ohio the maximum allowable percentage of wear may be as high as 10 or 12 per cent for limestones, 15 or 20 per cent for slag and 25 per cent for sandstone. In such cases the gradation calls for larger size particles; all passing a $5\frac{1}{2}$-in. and retained on a $2\frac{1}{2}$-in. screen for the coarse stone.

The top course is usually composed of harder stone, but if a bituminous surface treatment or bituminous surface course is to cover the top course, soft stone is sometimes permitted. Usually the top course is stone having a percentage of wear of not more than 5 or 6. The gradation of the coarse stone for this surface is between a 4-in. and $2\frac{1}{2}$-in. screen in Ohio, and between a $2\frac{1}{2}$-in. and $1\frac{1}{4}$-in. screen in Massachusetts, thus showing considerable variance in practice.

Lime Rock, Marl and Caliche.—These materials are soft—in fact so soft that wear tests are meaningless. They consist principally of carbonate of lime, of which more than 50 per cent is desirable. The best bases of Florida lime rock contain more than 70 per cent of carbonate of lime. Small stones and sand are not undesirable. A small percentage of clay, less than 10 per cent, is tolerated in caliche. When power rolling and watering are used in the shaping and bonding process, Florida permits all material to pass a $3\frac{1}{2}$-in. screen with not less than 30 per cent retained on a $\frac{3}{4}$-in. screen. In some of the western states which use caliche, smaller sizes all passing a $1\frac{1}{4}$-in. or 1-in. screen are specified when the surface is to be traffic bound without watering.

Miscellaneous Materials such as stone screenings, mine chats, iron ore topsoil, stamp sand, and others, are fragments of hard durable material. In size they all pass a $1\frac{1}{2}$-in. or 1-in. screen and may or may not contain a high percentage of material passing a $\frac{1}{4}$-in. screen or No. 10 sieve. The fines should preferably be non-slaking in character.

CONCLUSIONS

1. Materials which are adaptable and suitable for untreated surfaces and which can be improved later by the addition of other types of surfacing include stone, slag, gravel, lime rock, marl, caliche, chert, shale, disintegrated granite, shell, sand-clay and volcanic cinders.

2. Non-slaking binders for untreated surfaces, such as disintegrated limestone and stone screenings, are more satisfactory than clay or loam; relatively small proportions give best stability.

3. Stability of surface appears to be increased when the particles are angular in shape, but the wear of rubber tires is increased thereby.

4. Soft aggregates can be used more economically in a base course than in a surface course, but base courses of soft aggregate give reasonable service when covered by a relatively thin surface of hard and durable aggregate.

5. Surfaces constructed or maintained by blading or by dragging should be composed of aggregates passing a 1-in. or $\frac{3}{4}$-in. screen, and the immediate surface, to a depth of $\frac{1}{2}$-in. when used as a surface "mulch," should all pass a $\frac{1}{2}$-in. screen.

6. Present practice shows that, in general, quality and gradation of aggregates for low-cost untreated road surfaces are determined by usage and field service rather than by standard tests in either the laboratory or field.

7. There is now a need for the development of simple standard tests for determining the suitability, proportions, gradation and other characteristics of aggregates for untreated surfaces, dual bituminous surface treatments and mixed-in-place bituminous surfaces and for premixed-laid-cold bituminous surfaces.

MINERAL AGGREGATES FOR SEWAGE DISPOSAL AND FOR RAILROAD BALLAST

BY HERBERT F. KRIEGE[1]

SYNOPSIS

In the discussion of mineral aggregates as applied to sewage disposal practice, both trickling filter and contact beds are considered. The geographic distribution of these plants, their size and amounts of aggregate involved and the population served are briefly discussed. Under types of aggregates, the characteristics and behavior in practice of the three most widely used materials, namely, broken stone, slag and gravel are considered in detail.

Consideration is given to the factors affecting the life and behavior of disposal media such as handling, placement, climate, plant life, etc. In this way the conditions are dealt with which tend to cause disintegration, cementation and clogging in the beds. The corresponding effects of the life of the beds are measured along with other factors of operation which may seriously affect the efficiency of the disposal systems.

The history of several beds representing typical behavior of the various aggregates is included. The tests pertaining to the character of the media are fully covered, particularly the several methods for determining the soundness of aggregates. In view of the lack of uniformity in the interpretation of these tests as well as their procedure, a few definite suggestions are made on this phase of the subject. The behavior of some natural aggregates in the presence of certain questionable or deleterious minerals is discussed in the light of recent investigations. The economical considerations in the use of types of aggregate and the tendencies observed in the development of these forms of sewage treatment are brought out. Recommendations are made concerning the production of suitable aggregate for trickle filters and contact beds.

The treatment of aggregates as railroad ballast is based on a survey of the experience of railway engineers with the several commonly used ballast materials such as broken stone, gravel, slag, cinders, burnt clay, etc. The resistance of each type of ballast to impact and weathering, and its effect upon vegetative growth, ties and rails, dusting and proper drainage is pointed out. The mineral aggregates are classified according to their excellence in these several qualities —durability, availability, ease of placement and tamping, appearance of bed, rigidity, etc. It is pointed out that the interests of both the consumer and the producer of mineral aggregates are parallel in the development of improved aggregates from a careful consideration of the characteristics of each type and the conditions imposed by usage in the special fields.

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[1] In Charge of Tests, France Stone Co. Laboratories, Toledo, Ohio.

I. SEWAGE DISPOSAL AGGREGATES

With the increase in population about many centers the welfare of the growing communities depends upon the proper control of sanitation and hence to a great extent on effective sewage disposal. The types of disposal systems most widely used in this country are those employing contact and trickling filter beds. In both of these types, mineral aggregates are used as media for the beds. It is the purpose of the first part of this paper to discuss the characteristics and behavior of the various materials utilized in this connection in terms of conditions imposed upon them by usage.

The earlier types of sewage disposal systems, other than mere dilution in large streams or bodies of water, consisted of areas of sand or other small material over which the sewage was allowed to flow intermittently, thus exposing it to greater oxidizing and bacterial activities than it received upon slight dilution. As the quantity of sewage increased, the contact process was improved so as to consist of a well-defined area provided with good under-drainage and containing a mass of grosser mineral particles such as broken stone, slag, gravel, cinders, etc. The system as now used permits the slow filling of this contact bed with sewage which has previously had its heavier solids settled out, the sewage remaining in contact with the medium of the bed for a period of time, after which it is allowed to drain away through the under-drainage system. During this time of contact, a certain degree of stability has been reached through the partial oxidation of the organic substances contained in the sewage by the bacterial colonies lodged on the surfaces of the medium.

The contact method, though by no means discontinued, is giving way to the trickle filter bed system. This consists of an enclosed bed of aggregate, usually stone or slag, over which the sewage, after settling or treatment in the Imhoff tanks, etc., is sprinkled by means of sprays or traveling distributors. The sewage thereby receives some aëration before it comes in contact with the bacterial life on the medium. At all times in a properly functioning trickle filter bed, the large voidage provides for sufficient aëration to keep the bacterial colonies and sewage supplied with the oxygen demanded in the biochemical processes. During the time interval required by the sewage to trickle down the aggregate mass to the under-drain, a sufficient stability has been effected in the sewage to permit its passage directly into even small streams without creating a nuisance. The capacity of the trickle filter system is quite large, and hence serves in congested areas as a satisfactory and efficient sewage disposal method.

The distribution of the trickle filter systems through the United States is rather wide. According to the 1927 Progress Report to the American Society of Civil Engineers the Filtering Materials Committee of the Sanitary Engineering Division[1] found that in 19 states east of the Mississippi River a total of 142 plants was in use serving a population of 2,800,000. West of the Mississippi River, nine states reported a total of 246 plants serving a population of 1,240,000, making a combined total of 388 plants serving 4,040,000 people in the 28 states reporting prior to April, 1927. Since that time a survey by the author has added 97 plants, serving an additional population of 754,950, making a total reported up to April, 1929, of 485 plants in 34 states. The distribution of these plants is stated more specifically in Table I.

TABLE I.—INSTALLATIONS OF SEWAGE FILTER PLANTS REPORTED WITH APPROXIMATED POPULATION DATA.

State	All Plants Reported		State	All Plants Reported	
	Number	Approximate Population		Number	Approximate Population
STATES EAST OF THE MISSISSIPPI RIVER					
Alabama[a]	1	80 000	New York	25	214 400
Connecticut[a]	1	6 000	North Carolina	12	60 350
Delaware	2	4 100	Ohio	28	590 000
Georgia[a]	4	385 000	Pennsylvania[a]	14	76 000
Illinois	34	453 200	Rhode Island	2	13 400
Indiana[a]	5	30 000	Tennessee	7	27 100
Kentucky[a]	2	53 000	Virginia[a]	6	12 000
Maryland	10	615 600	West Virginia	2	6 400
Massachusetts[a]	4	304 000	Wisconsin	13	70 500
Michigan	9	96 400			
Mississippi	1	11 000	Total east of Mississippi River	200	3 212 050
New Jersey[a]	18	103 600			
STATES WEST OF THE MISSISSIPPI RIVER					
Arizona[a]	2	13 700	Oklahoma	45	318 000
California[a]	14	49 700	Oregon	1	10 000
Colorado[a]	4	20 500	South Dakota[a]	9	54 700
Iowa[a]	38	94 300	Texas[a]	122	718 000
Kansas	19	82 600	Washington	1	20 000
Minnesota	6	45 300			
Missouri	22	131 600			
North Dakota	2	24 500	Total west of Mississippi River	285	1 582 900
Grand Total				485	4 794 950

[a] Data taken from the April, 1927, Progress Report of the Committee on Filtering Materials, Sanitary Engineering Division of the American Society of Civil Engineers.

The volume of mineral aggregates entering into the trickle filter beds in the states reported amounts to 1,500,000 cubic yards. Therefore, these sewage disposal systems constitute a considerable market for aggregates, especially since the materials used are confined within rather close limits to one size.

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[1] *Proceedings*, Am. Soc. Civil Engrs., April, 1927, Part I.

The aggregates which have come into most general use in both contact and trickle filter beds are broken stone, including both sedimentary and igneous rocks, blast-furnace slag and occasionally gravel, cinders, broken tile, broken retorts, etc. This variety in types of aggregate has been brought about by the desire to find some medium which would give a large effective surface for lodgment of bacterial life and yet would be an economically possible material. Experience has shown that certain of these aggregates are distinctly superior to others as to durability and general effectiveness.

In considering the requirements which the medium in a trickle or contact bed must meet, the conditions imposed by such usage need discussion. In practice the medium is intermittently wetted with the sewage and allowed to drain. This facilitates any disintegration or attack which hydration and oxidation under moist conditions might bring to pass. The action becomes particularly severe when during the winter months the aggregate is repeatedly frozen while wet and thawed by subsequent applications of the warmer sewage. In the trickle bed a dosing period of three to six minutes followed by a rest period of ten or fifteen minutes is not uncommon. This means that in twenty-four hours the filter bed medium may be alternately frozen and thawed nearly one hundred times, provided the temperature of the air is well below freezing. This becomes perhaps the most severe test which mineral aggregate is called upon to withstand in industrial practice. The properties of mineral aggregates which will enable them to withstand these conditions of intensive weathering will be considered separately.

The rapid growth in all lines of highway and building construction has required a more rigid classification of materials suitable for the work. The characteristics most commonly specified and tested for in mineral aggregates are hardness, toughness, abrasion, absorption, weight per cubic foot and soundness. It is quite natural, then, that the group of engineers interested in sewage disposal practice should call upon these same specifications and test procedures to determine the suitability of media in sewage treatment. Some questions may be raised, however, as to the advisability of this plan since it may lead to a false conception of what consistutes the best material for this specific purpose. It is perhaps advisable that as much thought be given to color, uniformity, crystallinity, fracture, cleavage, and the screen analysis with the percentage of voids of aggregates in the selection of the material. Roughness has been discussed though without any definite recommendations being made since no easy measure for the degree of roughness has been reported.

Hardness.—The hardness as commonly determined for aggregates is the resistance to wear or abrasion under standard conditions as prescribed in the procedure for the Dorry hardness test. The occasions on which either contact or trickle filter media are called upon in practice to show resistance to wear are few, about the only movement which the material passes through being that involved in the placing of the material in the bed. Hardness is no criterion of the durability of aggregates, particularly in the sedimentary stones which find quite general use in sewage disposal work. One of the hardest of the common minerals associated with the limestone deposits of this country is the practically pure silica mineral known as chert. This has a hardness approximating that of quartz, yet one form of it is a readily disintegrating material upon exposure to weather or sewage treatment. Likewise the hardest limestones are very often the ones that show the greatest tendency toward disintegration when exposed. This applies equally well to gravels composed largely of limestone particles. In the case of blast-furnace slag, which may be either dense, medium dense or honeycombed, the dense variety is naturally the hardest and yet this is more apt to show splitting either in actual or in experimental freezing and thawing or in the sodium sulfate test than is the medium dense or the honeycombed slag.

Toughness.—Since toughness is taken as the measure of the resistance of the material to impact, the same conditions referred to for hardness may be applied in this case. Practically the only impact which the material receives is during the placing. Aggregate of but moderate toughness can easily withstand that amount of impact. Likewise aggregates of great toughness often do not show any greater resistance to the disintegrating action of wetting and drying, freezing and thawing, than do those of lower resistance to impact. It does not seem advisable therefore to discriminate against an aggregate of lower toughness value for this reason only.

Abrasion.—The resistance to wear of one piece of aggregate against others and against the walls of the containing vessel is sought for in the standard abrasion test. This characteristic is somewhat allied to hardness and toughness and what applies to them applies also to abrasion. Argillaceous limestone and some chert may be mentioned in this connection since their abrasion losses are very low, yet their suitability for filter bed use may be very seriously questioned.

Absorption or Porosity.—Density in the mineral aggregates entering contact and trickle filter beds has been sought for quite generally, on the supposition that material which absorbs the least water or has the lowest pore space will be the one longest surviving the disintegrating action of freezing while wet. Woolf[1] has recently shown that when all types of naturally occurring aggregates are considered, igneous and sedimentary stones being included, the resistance to disintegration by five sodium sulfate immersions is somewhat in relation to the density of the stone. Unpublished data secured in the laboratory of the France Stone Co. show that this does not hold true when applied to limestones and dolomites and their associated minerals, particularly when the sodium sulfate test is carried to twenty cycles. The relation between the soundness as determined by these tests and the absorption on several hundred samples of such materials is highly irregular.

Weight per Cubic Foot.—From an economic standpoint at least, the weight per cubic foot has some bearing on the use of a certain medium. The purpose of the medium is simply to provide a sufficiently large surface on which the bacterial life may lodge to effectively alter the sewage in contact with it. Hence, the smaller weight of material used to provide this surface, the lower will be the cost, other things being equal. In this way one aggregate of lower density or higher porosity, even though it shows a higher abrasion loss, lower toughness and lower hardness, may be the proper one to use. Since a difference in weight of 500 lb. per cubic yard is not unusual between moderately porous limestones and dense trap rocks or gravel consisting largely of igneous material, it can easily be recognized that the unit weight is a definite factor in the economics involved. On comparing a honeycombed slag with the dense stone, the difference will be even greater.

Color.—While it may appear that color has little if any bearing on the characterization of the material for its use in sewage disposal practice, certain observations which may be made by visual inspection will prove helpful in the selection of the right material. Color is often an index to the chemical composition or the presence of certain minor constituents which may have a bearing on the general properties of the material. Usually the yellows, reds and browns indicate that the material has already been subjected to oxidizing conditions, while the blues, greys and blacks denote the presence of unoxidized substances such as carbon or iron compounds. Material already in the oxidized state may be expected to undergo less change upon further exposure than one in which this oxidation has not yet occurred. The presence of small amounts of silt, clay, beidellite and organic films can be detected more easily by color or luster differences than by any other simple means. In the case of limestones in the central states, some strata having a dull lusterless ash-gray color on fresh fracture may be regarded with suspicion, regardless of their other physical and chemical properties, since usually they show very little resistance to weathering action.

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[1] D. O. Woolf, Jr., *Public Roads*, Vol. 8, December, 1927, p. 225.

Uniformity.—The lack of uniformity in composition, kind of material, structure, and texture may be taken as an evidence of lessened resistance to the action of weathering. In the freezing-and-thawing experiments or other accelerated weathering tests the first evidences of the disintegrating tendencies seem to occur at points of nonuniformity in the material. Laminations, sutures, imbedded accessory crystals or masses, and inclusions, be they of microscopic or macroscopic size, appear to be the weakest portions of the defense against disintegration of the material. Blast-furnace slag which has every reason to be uniform in composition is a good example of how this uniformity beneficially affects resistance to freezing and weathering in general. These remarks should not be construed to mean that any material which is uniform is necessarily stable and resistant to disintegrating forces. Lack of uniformity however must be given due consideration when characterizing a material for use in exposed conditions. Reference should be made again to some chert whose tendency to disintegrate is so pronounced while its most closely allied mineral, namely, flint, is a common synonym for durability. In a recently published article by the author[1] the instability of disintegrating chert is attributed more to lack of uniformity in mineral structure and to enclosed crystalline foreign masses than to any other observed property.

Crystallinity.—Whether a material is holocrystalline, grossly or finely crystalline, crypto-crystalline or amorphous does not seem to have any great bearing upon its behavior in trickle or contact bed use. However, when within a single piece of aggregate the crystallinity changes definitely, such as a zone of gross crystals within a finely crystallin or amorphous mass, or an amorphous clayey seam within a well crystallized mass, such structures encourage disintegration at that point. Since in the preparation of crushed mineral aggregates for the market the weaknesses of such structures are usually found by the fracture stresses during the crushing operation, not many of these weak planes persist in the finished pieces.

Fracture and Cleavage.—The fracture and cleavage habits of the commercially prepared products have some bearing on their fitness as filter media. The tendency toward slabby pieces is undesirable since this tends to produce a low voidage in the resultant bed due to overlapping of these slabby pieces in layers like shingles of a roof. Those fracture habits which leave the prepared aggregate with angular shapes and irregular surfaces are most desirable. In this property, slag, trap rock, and granite are most commonly in the desired class. Sedimentary stones free from shaley or slatey material likewise fall into this category.

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[1] H. F. Kriege, "The Stability of Chert," *Rock Products*, Vol. XXXII, No. 9, April 27, 1929, p. 50.

Roughness.—Since roughness largely determines the amount of surface which a piece of aggregate offers, this property should be taken into account in the selection of a medium for purposes utilizing its surfaces. Experiments conducted at Philadelphia[1] to show the relative efficiencies of aggregates varying widely in their shape and roughness brought out the fact that the rougher the material the more efficient it was in the oxidation and stabilization of the sewage influent. The extremes of roughness in materials available commercially are well rounded gravel on the one hand and honeycombed slag on the other. The opinion has been expressed by some sanitary engineers that slag offers too great a roughness, that this roughness tends to hold the bacterial colonies and collected solids too long so that clogging of the bed is produced. The Progress Report of the Committee on Filtering Materials[2] contains some data pertaining to the effect of type of medium on the clogging tendencies. These have been rearranged in tabular form shown in Table II.

TABLE II.—RELATION OF TYPE OF MEDIUM TO TRICKLE BED CLOGGING TENDENCY.

MATERIAL	NUMBER OF PLANTS REPORTING	AVERAGE TOTAL YEARS OF SERVICE	PERCENTAGE SHOWING CLOGGING
Limestone	51	8	48
Granite, trap	29	7	23
Blast-furnace slag	11	8	18
Gravel	8	6	50

It appears, therefore, from actual experience gained with trickle beds operated in widely separated districts that clogging is not so common with highly irregular and rough surfaced material like slag as with the smooth, rounded gravel.

In February, 1929, the author saw the larger of these slag beds reported in the table above as having clogged. At the time of inspection (during a rest period) it was entirely clean, practically the only evidence that it was not freshly placed slag being a general iron stain covering the surfaces of the slag pieces in the zone of the sprays. This stain was apparently due to the iron compounds contained in the influent. A rest period of several months had completely relieved this bed of its clogging material even though it had been heavily overloaded for twelve years with an industrial and domestic sewage which was difficult to treat. Mr. C. C. Hommon,[1] in charge of the Canton sewage disposal plant, after examining all the known trickle beds using blast-furnace slag as the filtering medium, reported 29 on which no clogging had occurred, while 12 showed clogging or clogging tendencies. However, the tendency to clog in the latter cases could be explained in every instance (1) by poor sizing of the material placed in the bed; (2) by excessive loading without rest periods, or (3) by the poor condition of the influent. On the other hand, perhaps the largest gravel bed now in use was observed by the author during February, 1929, and found to be in a very dirty condition, indicating that this may occur even on a bed composed of smooth rounded pieces. Clogging of trickle beds is therefore due to some factor other than the roughness of the surface of the medium.

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[1] Report Sewage Testing Station, page 120; also William Metcalf and H. P. Eddy, "American Sewerage Practice," Vol. 3, p. 555 (1916).

[2] *Loc. cit.*

TABLE III.—RELATIVE ROUGHNESS VALUES OF SOME DOLOMITIC LIMESTONES.

(Based on a spherical surface as unity.)

	DESCRIPTION	ROUGHNESS VALUE
Limestone A	Almost cubical shape with smooth surfaces	1.39
Limestone B	Fairly cubical shape with smooth surfaces	1.60
Limestone C	Fairly cubical shape with rougher surfaces	1.84
Limestone D	Fairly cubical shape with rougher surfaces	1.87
Limestone E	Irregular fracture, angular shape and fairly smooth surfaces	2.01
Limestone F	Irregular fracture, angular shape and irregular surfaces	3.01
Limestone G	Irregular fracture, angular shape and irregular surfaces	3 10

Little has been definitely learned about the relative roughness of different filtering media. In order to arrive at some fair understanding of this property in limestones known to be different in their fracture and cleavage habits, the following tests were conducted in the France Stone Co. laboratories. It had been previously shown[2] that the fracture habits of limestones persist into the smaller sizes when the stone is crushed. In this way it is possible to study the roughness of any convenient size of material, say of No. 4 to No. 6 sieve size, and use that as a standard for either the larger or the smaller fragments.

The procedure consisted of determining the amount of material dissolved in dilute hydrochloric acid under controlled conditions of temperature, acidity, agitation and time, from a known area of polished surface of the limestone sample. This gave the specific solubility rate for that kind of material per unit surface. This same weight of material of No. 4 to No. 6 size was run under identical conditions. Since dissolution takes place only at the surface of the dissolving substances, the different amounts dissolved from the polished and the naturally rough pieces were in relation to the differences of the surfaces exposed or in other words to the roughness. By comparing the amount of material dissolved from a given weight of the stone particles with the amount that would have dissolved had these particles been true spheres, a numerical ratio of roughness (including both shape and surface irregularities) was obtained. Some of the values found in this investigation are given in Table III, to indicate how widely the degree of roughness may vary even within the same class of material in this case dolomitic limestone.

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[1] Unpublished report to the National Slag Assn.

[2] H. F. Kriege, "Factors Affecting the Dissolution Rates of Limestones in Acid Media," *Rock Products*, Vol. 32, No. 10, May 11, 1929, p. 73.

The method is not applicable to slags and other silicates.

Soundness.—Since the characteristic which most largely determines the usefulness of an aggregate for filter bed use is its ability to resist breaking down under the actions and forces imposed by service, several laboratory methods for predicting the behavior of materials in use have been developed. The more successful ones consist of (1) actual freezing and thawing at relatively short intervals and (2) a drying and soaking process in solutions such as sodium sulfate and sodium chloride in which the destructive effect of the crystal growth of the solution contained in the pores of immersed material is used as a measure of its resistance to actual freezing conditions. A more satisfactory method of testing, where time permits, is to expose the material in question upon the surface of some trickle bed in operation and to observe its behavior through a period sufficiently long to warrant conclusions being drawn. This method is being used by several sanitary engineers and aggregate producers.

From a laboratory standpoint, the sodium sulfate test for soundness is perhaps the most acceptable since it involves but little apparatus, labor, or space and results can be obtained within several weeks. An attempt has recently been made by the Committee on Filtering Materials of the A. S. C. E. to standardize the procedure for this test sufficiently to permit the same type of behavior to be found with a material sent to different laboratories for testing. While the results of this work have not yet been made public, it is believed that such an effort is well worth while and that it will show that no consequential variations result if the several laboratories observe the outlined procedure with fair accuracy. The procedure as followed in the France Stone Co. laboratories consists of immersing the samples, which have been dried for four hours at 105° C., in a solution of sodium sulfate which has previously been saturated at 30 to 35° C. and allowed to cool to room temperature. One cycle consists of 20 hours' immersion and 4 hours' drying, the specimens being examined immediately after removal from solution for any signs of disintegration, checking, flaking, cracking, etc. The number of cycles is usually 20, this number having been arbitrarily set as indicating a sufficient degree of resistance to expect long life in actual practice.

The materials which show distress early in the sodium sulfate test are (1) shales, (2) slatey material, (3) stone with carbon seams, (4) limestones with high non-carbonate content and (5) limestones containing chert of the kind which tends to disintegrate. Usually the destruction of these occurs before ten cycles are run, often before the fifth cycle. Blast-furnace slag, granite, trap rock and other igneous materials are noteworthy in their ability to resist the sodium sulfate test. Likewise these same materials show little disintegration in use in the filter beds. Limestones with their wider variations in composition, structure and association show all ranges of resistance to this test and to actual weathering.

There is considerable confusion as to what types of disintegration are most serious to the effective service of the filtering medium. Some fear the crumbling type of disintegration, others the splitting into slabby pieces. Obviously that type of disintegration which provides material either (1) large enough to cause void reduction within the mass of the bed or (2) small enough to be retained by the organic matter near the surface of the bed would tend to lower the efficiency of the filtering medium. In order to describe more fully the types of disintegration which may occur in stone samples, the following key is given, which has been used in recording all the varieties of distress noted in the author's soundness tests. The amounts of material broken off during all of these forms of failing are indicated in grams.

A	Horizontal split into 2 pieces.
*A*3, 4	Horizontal split into more pieces.
B	Vertical split into 2 pieces.
*B*3, 4	Vertical split into more pieces.
H	Horizontal chips from 1 piece.
*H*2, 3	Horizontal chips from 2 or 3 pieces.
V	Vertical chips from 1 piece.
*V*2, 3	Vertical chips from 2 or 3 pieces.
Ban	Banding failure.
C	Imminent checking and splitting.
Ch	Failure around cherty inclusions and limestone contact.
Ch_v	Failure across cherty inclusions and limestone contact.
Cr	Failure of crystals at contact.
D	Complete disintegration by crumbling, etc.

*D*1, 2	Number of pieces showing complete disintegration by crumbling, etc.
F	Flaking—scaling on any side, horizontal or vertical.
I	Infilling crumbling.
L	Laminations failing.
R	Broken pieces retained in the series to observe further breaking action.
S	Surface solution effect.
Sc	Scaling at surface.
Sh	Shale failing.
Su	Sutures failing.
OK	Passed through individual test successfully.

Considerable work has been done by Orton,[1] Staley,[2] Withey,[3] Schlick[4] and other investigators with mineral aggregates and other non-metallic structural materials to find the relationship between the effects of freezing and thawing, either in the natural or the accelerated state, and the laboratory results of sodium sulfate, sodium chloride, magnesium sulfate, alkali and boiling and drying tests. No fixed relationship has been observed, however, the values of one sulfate cycle ranging from less than one to nearly one hundred freezing cycles with different types of materials or conditions of testing. Perhaps the most logical method is that suggested by G. B. Gascoigne,[5] in which the behavior of an untried material is predicted from the comparison of its accelerated weathering resistance with that of a material known to be satisfactory both in test and service. Nevertheless it is becoming more evident as our experience with filter media increases that a material which will withstand 20 cycles of the sodium sulfate test as is most widely followed to-day,[6] can be expected to stand up well for several decades under the severe conditions imposed by actual service in trickle beds.

The factors directly affecting soundness of natural mineral aggregates of sedimentary origin are lack of chemical or physical uniformity within the specimen, more specifically existence of certain associated minerals such as shale, pyrite and some cherts, and the presence of considerable amounts of clayey material, beidellite, and similar hydrous alumino-silicates. The usually considered physical properties of hardness, toughness, abrasion, etc., play but minor parts in determining the life of filter material as was pointed out in the previous discussion. The disintegration of granites and other igneous materials is so slow that the development of their factors of degradation is unnecessary in this connection.

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[1] Edward Orton, Jr., "A Study of the Proposed A.S.T.M. Tentative Specifications for Building Brick and a Correlation of Their Requirements with Sodium-Sulfate Treatment and Actual Freezing," *Proceedings*, Am. Soc. Testing Mats., Vol. XIX, Part I, p. 268 (1919).

[2] H. F. Staley, *Transactions*, Am. Ceramic Soc., Vol. 18, p. 642 (1916).

[3] M. O. Withey, *Proceedings*, Fourth Annual Meeting Highway Research Board (1925).

[4] W. J. Schlick, *Bulletin 89*, Engineering Experiment Station, Iowa State College (1928).

[5] G. B. Gascoigne, unpublished report on trickler filter media.

[6] U. S. Department of Agriculture *Bulletin 1216*, p. 8 (1924). Revised form, p. 206 (1928).

TABLE IV.—FIELD OBSERVATIONS AND LABORATORY TESTS ON SLAG USED AS TRICKLE BED MEDIA.

Plant Location	Operated Since	Disintegration of Material	Weight per Cubic Foot, Loose, lb.	Absorption, per cent	Apparent Specific Gravity	Voids, per cent	Abrasion, per cent	Soundness Behavior O.K. after 20 Cycles unless otherwise stated	Chemical Composition						
									Silica, per cent	Alumina, per cent	Ferric Oxide, per cent	Manganous Oxide, per cent	Calcium Oxide, per cent	Magnesium Oxide, per cent	Sulfur, per cent
Akron, Ohio	1916	Little evidence of true disintegration	60	2.6	1.98	44.0	16.1	O. K.	33.10	12.22	1.08	0.88	40.23	7.25	0.64
Amherst, Ohio	1928	None	68	3.8	2.17	49.8	7.0	O. K.	36.38	11.24	0.70	1.25	43.04	5.51	1.16
Bay Head, N. J	1916	None	55	5.4	1.87	45.1	16.8	O. K.	37.04	9.38	0.76	0.79	38.75	12.36	0.54
Battlewood, N. J	1922	None	58	2.7	2.04	46.5	17.7	O. K.	35.60	12.70	0.50	0.94	37.58	10.97	0.83
Bolivar, N. Y	1928	None	61	4.2	2.07	45.0	14.8	O. K.	36.86	11.70	1.88	1.06	42.32	4.63	1.20
Camp McClellan, Ala	1918–1919	Little disintegration	63	3.2	2.04	42.6	10.8	O. K.	38.16	10.93	2.41	0.53	36.93	6.31	0.62
Canton, Ohio	1925	None	58	3.8	1.96	44.4	17.8	O. K.	34.28	13.30	0.70	1.07	40.05	6.60	1.31
Colwick, N. J	1926	None	59	6.2	1.88	42.0	16.3	O. K.	37.56	11.49	1.01	0.86	30.87	14.77	1.06
Dixmont, Pa	1910	None	63	2.0	2.12	44.0	8.8	O. K.	36.38	12.91	1.01	0.76	42.71	3.74	0.97
East Aurora, N. Y	1923	Small amount splitting	62	3.8	1.96	41.9	11.8	O. K.	36.50	16.78	0.50	0.85	37.71	3.23	1.13
Edgewater Mines, Ala	1917	No true disintegration	51	5.1	1.85	47.9	17.9	O. K.	36.64	11.61	0.51	0.48	38.98	8.48	0.33
Edgewater Mines, Ala	1925	None	51	5.1	1.85	47.9	17.9	O. K.	36.64	11.61	0.51	0.48	38.98	8.48	0.33
Erie County, Pa	1924	None	66	1.5	2.29	47.5	13.9	O. K.	36.72	9.72	0.50	1.14	43.61	8.54	0.90
Fairfield, Ala	1914	No late disintegration	..	2.6	2.29	....	11.0	[a]	38.60	10.20	1.34	0.76	41.48	5.88	0.68
Glassboro, N. Y	1926	None	57	4.9	1.89	44.0	15.2	O. K.	36.06	12.19	0.77	1.64	34.20	12.44	1.14
Haddonfield, N. J	1920	None	56	6.4	1.93	46.0	18.8	O. K.	36.00	13.14	0.76	0.78	34.11	12.34	0.68
Hammonton, N. J	1914	None	63	7.8	2.04	43.4	21.3	O. K.	41.60	12.79	1.01	0.85	29.80	11.63	0.65
Ishkooda Mines, Ala	1921	None	62	2.0	2.16	47.3	12.5	O. K.	38.30	11.21	0.51	0.73	40.77	6.45	0.52
Leetonia, Ohio	1925	Little disintegration at surface	59	3.9	1.85	41.9	19.6	O. K.	32.62	14.44	0.76	0.62	43.46	2.67	0.94
Maple Shade, N. J	1927	Very little disintegration	58	5.8	1.88	42.8	16.8	O. K.	36.28	12.43	0.63	1.37	33.88	13.85	1.52
Media, Pa	1924	None	57	4.0	1.87	43.4	17.8	O. K.	38.24	12.10	0.63	1.24	36.30	10.09	1.45
Johnson City, Tenn	1924	None	75	2.7	2.31	40.3	14.2	O. K.	39.18	5.58	0.64	0.60	34.78	16.14	0.49
Muse, Pa	1925	None	64	4.0	2.29	48.9	14.7	O. K.	34.34	13.73	0.57	0.83	42.13	3.35	1.40
Oaklyn, N. J	1915	None	57	5.7	1.93	44.3	17.2	O. K.	38.16	11.72	0.70	0.72	34.86	10.82	0.49
Pitman, N. J., Number 1	1917	No true disintegration	61	3.7	1.98	43.3	15.3	O. K.	36.42	12.75	0.89	1.02	35.23	11.40	0.82
Quakertown, Pa	1927	None	56	5.7	1.99	47.6	16.8	O. K.	35.01	12.92	0.38	1.09	35.90	12.71	1.11
Birmingham, Ala	1926	None	65	2.5	2.14	43.0	7.6	O. K.	39.04	10.40	0.70	0.58	39.69	6.08	1.01
South Coatesville, Pa	1916	None	64	4.1	2.00	40.7	24.7	O. K.	33.68	12.10	2.52	0.39	42.61	2.16	0.91
Buffington, Ind	1926	None	73	2.2	2.40	44.5	8.4	O. K.	39.12	11.14	0.72	1.07	42.84	3.74	1.24
Universal, Pa	1927	None	67	2.6	2.35	46.1	11.7	O. K.	34.92	13.23	0.69	0.59	42.49	5.10	0.94
Birmingham, Ala., Jefferson Co.	1919	None	..	3.8	2.00	....	9.0	O. K.	35.34	11.41	1.41	0.56	41.84	7.99	0.49
Westmont, N. J	1921	None	58	5.3	1.84	41.6	16.4	O. K.	35.70	13.41	0.33	1.08	38.79	7.35	0.80
Woodbury Heights, N. J	1922	Marked evidence of disintegration	55	4.7	1.91	46.3	18.3	[b]	34.04	13.01	0.63	0.62	40.78	10.01	0.72
High			75	7.8	2.40	....	....	....	41.60	14.40	2.52	1.64	43.61	16.14	1.52
Low			51	2.0	1.84	....	....	....	32.60	5.58	0.33	0.39	30.87	2.67	0.33

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[a] 1 small chip at eighteenth cycle.

[b] 1 piece in two at twelfth immersion, 1 piece in two at fourteenth immersion, 1 piece in 10 parts at seventeenth immersion and 1 piece in 7 parts at eighteenth immersion. Contained small piece of coke impregnated with iron.

Sometimes one finds a tendency to specify the chemical composition or some physical property for blast-furnace slag if it is to be considered for sewage disposal bed purposes. Such conditions as low lime, low sulfur, or low iron content are called for, with some thought being given to the density of the material. Slag, by the very reason of its existence, is made to conform to a definite chemical composition within narrow limits. It does not therefore seem the part of wisdom to arbitrarily set standards for slag if its use in a special field like sewage disposal is being considered. Rather it should be taken as it is if tests and use have demonstrated its suitability for that service. In Table IV are found data secured by Fred Hubbard of the Standard Slag Co., Youngstown, Ohio, on slags that were collected by C. C. Hommon from trickle and contact beds in service. It should be observed how little variation there is in these slag samples in both composition and in physical properties, and how uniformly well they have withstood the conditions of actual service and the subsequent 20 sodium sulfate cycles. These figures amply support the statement that the slight variations found in the chemical and physical properties of commercially prepared slag are of no significance to the durability of the material and its quality for sewage disposal beds.

The sole function of the medium in a trickle filter bed is to provide a large amount of well aërated surface. To maintain proper circulation of air within the bed, the medium should have a high percentage of large voids. Hence, if a material has a large specific surface, its excellence for this use will depend upon the factors controlling its voids. These are sizing, cleanliness and durability. In this day when the washing of commercial aggregates is so common, it should not be difficult to secure material of the desired cleanliness. Likewise proper sizing should be entirely within the control of the engineer in charge of the job and the producer of aggregates. Judging from beds now in operation, sufficient attention has not been paid to this matter of sizing. The quantity of fine material below that specified not infrequently runs to 15 to 20 per cent of the total. This very definitely reduces the voidage of the bed and hence the air capacity and promotes clogging. According to statements of state health officers of several states in the Great Lakes region, improper sizing has contributed more often to the poor behavior of beds in these states than has disintegration of the material after placement.

A tendency to raise the minimum size of material is evident. Taking the sizes specified for some 50 plants into consideration, the average size of aggregate for the upper part of the bed is found to have changed with time in the following manner: previous to 1910, 1.1 to

2.8 in.; 1910 to 1915, 1.2 to 2.5 in.; 1915 to 1920, 1.7 to 3.3 in.; since 1920, 1.8 to 3.3 in. It is logical to expect a continued increase in the smaller dimension since this will provide for larger voids and less opportunity for imperfect oxidation or for clogging.

Another evil attending poor sizing is the tendency for the fine material to concentrate in portions of the bed during placement unless special care is taken to prevent it. These areas invite trouble such as pooling and clogging since they offer greater resistance to the downward movement of sewage and the solids carried thereby. If the material has the property of cementation, this will be most apt to occur in such areas of concentrated fine particles. It is doubtful if cementation ever results from the accumulation of fine material broken down from the grosser pieces by disintegration.

Occasionally a trickle bed is reported to be giving unsatisfactory service. Usually the filtering medium is blamed for this condition. However it is wise to ask concerning the operating practices of the plant. There are several well-known cases of the unsatisfactory performance of sewage plants in which the trouble was not due to the aggregates. Beds of granite have clogged, pooled and flooded until cleaning was necessary, yet no characteristic of the stone contributed to this trouble. The only explanation lies in the improper operation of such beds. Overloading is the most frequent operation abuse. This factor probably accounts for more trouble than any other.

Poor screening and insufficient settling of the sewage permit excessive quantities of solid matter to pass upon the beds. The accumulation of this foreign material contributes definitely to the clogging of the filtering medium. Surface drainage carrying run-off sediment to the bed constitutes another source of trouble. Unless proper provision is made against it the filter areas may become littered with leaves and similar debris. Each of these operating factors may impair or destroy the efficiency of a trickle or contact bed otherwise in good condition.

As practical measures for producers of filter bed media, the following suggestions are offered:

1. Establish the durability of the material by laboratory tests (preferably sodium sulfate) and if possible by observing its behavior on the surface of a bed in operation through one winter. Field observation indicates that the disintegration of aggregate takes place almost entirely at the surface of the bed and to the greatest extent during the first year or two.

2. The inspection and testing of the stone quarry, slag bank, or gravel pit is not of as great importance as the testing of the prepared aggregate of the size desired for that use.

3. Cleanliness and close adherence to the size specified are of the utmost importance. Such attention and effort will involve a slight additional cost, but it greatly improves the quality of the material and makes it distinctly worth more to the consumer.

CONCLUSIONS

The conditions imposed by sewage disposal practice in which mineral aggregates are used in trickle filters or contact beds are very severe. However, aggregates of sufficient durability to be used in this practice are commercially available in most sections of the country. The function of the medium is to provide a large well aërated surface. To this end the proper sizing, cleanliness, placing, and the durability of the medium are of utmost importance. The first three of these factors can be controlled while the fourth can be predetermined with a sufficient degree of accuracy by soundness tests. Twenty sodium sulfate cycles with stone, slag and gravel aggregates indicate sufficient durability for satisfactory service as sewage disposal media. Many beds are now operating successfully in which the material will not withstand 20 cycles. The usually considered physical properties such as hardness, toughness, abrasion, etc., are of minor significance as criteria of the fitness of mineral aggregates in this field. Poor operation of a sewage disposal plant is frequently the cause of inefficient behavior of otherwise satisfactory trickle or contact beds.

II. BALLAST

One of the greatest fields for mineral aggregates is railway ballast. From the standpoint of tonnage involved it greatly exceeds the other demand for aggregates exposed to weathering conditions, namely as sewage disposal bed material. The conditions of exposure are less severe. The widespread distribution of the demand for ballast has called into play a variety of materials. Their behavior in practice is worthy of some consideration.

A good ballast material should be resistant to the impact of placing instruments and of the trains running over it. It should be weather resistant. It should provide a bed which drains well, retards or prevents the growth of vegetation, does not dust, has no injurious effect on ties and rails, is placed without too much difficulty, has good rigidity, presents a good appearance, and is readily available in quantity. The materials which have met these requirements more or less satisfactorily are washed gravel, trap rock, granite, limestone, blast-furnace slag, copper and lead slags, pit-run gravel, chats, cinders, burnt clay and shell.

Since many of the railways own sources of ballast, it is not possible to arrive at a close figure as to what amounts of ballast are purchased from the producers of commercial aggregates. The largest tonnage involved is gravel coming partly from commercial sources. Just how important ballast is as a market for blast-furnace slag is shown by the following figures. In 1920, 48 railroads in the eastern half of the

TABLE V.—BEHAVIOR OF BALLAST IN PRACTICE.

(Summarized expressions of railway engineers.)

	STONE	GRAVEL	SLAG	CINDERS	CHATS	CLAY	SHELL
Resistance to impact......	16*g*	19*g*	9*g*	2*g*	2*g*	..	..
	2*f*	8*f*	3*f*	7*f*	..	1*f*	3*f*
	..	..	..	5*p*	..	1*p*	..
Resistance to weathering..	1*vg*	..	1*vg*	..	..	..	..
	16*g*	18*g*	9*g*	3*g*	2*g*	..	1*g*
	2*f*	9*f*	3*f*	9*f*	..	..	2*f*
	..	1*p*	..	2*p*	..	2*p*	..
Drainage......	1*vg*	..	..	..	..	..	..
	16*g*	16*g*	11*g*	4*g*	1*g*	..	1*g*
	3*f*	13*f*	2*f*	6*f*	..	1*f*	2*f*
	..	1*p*	..	4*p*	1*p*	1*p*	..
Effect on ties and rails....	13*g*	24*g*	9*g*	5*g*	..	1*g*	2*g*
	5*f*	3*f*	3*f*	3*f*	..	..	1*f*
	..	1*p*	..	6*p*	1*p*	..	..
Effect on vegetation.......	14*g*	9*g*	10*g*	3*g*	1*g*	1*g*	2*g*
	4*f*	10*f*	2*f*	7*f*	..	..	..
	..	6*p*	..	4*p*	..	1*p*	1*p*
Effect on dusting.........	1*vg*	2*vg*	1*vg*	..	..	..	..
	13*g*	10*g*	11*g*	2*g*	2*g*	1*g*	..
	4*f*	6*f*	..	3*f*	..	..	1*f*
	..	4*p*	..	9*p*	..	1*p*	2*p*

NOTE.—*vg* = very good.
g = good.
f = fair.
p = poor.

TABLE VI.—COMPARATIVE VALUES OF BALLAST IN USE.

(Summarized expressions of railway engineers.)

				GRANULATED					
	STONE	GRAVEL	SLAG	SLAG	CHATS	SHELL	CLAY	SAND	CINDERS
Durability...	1.39	2.10	1.67	2.00	1.50	3.33	4.0	3.0	3.26
Availability..	1.98	1.62	2.40	1.00	4.50	2.00	5.0	2.0	1.88
Tamping....	2.50	1.49	3.33	2.00	1.00	1.67	3.5	3.0	1.87
Appearance..	1.49	2.14	1.54	2.00	2.50	2.33	3.5	3.0	3.02
Rigidity.....	1.34	2.22	1.29	2.00	3.50	2.33	4.5	3.0	2.96

NOTE.—The materials were placed in order of their excellence as first, second, third, etc., under each condition. The values shown are the averages of these numbers. The lowest value indicates the greatest excellence.

United States were using annually 1,440,000 tons of slag. In 1926, railroads in the midwestern states used nearly a million tons. One slag company during the same year furnished 575,000 tons for ballast. To the crushed stone industry ballast offers a still larger field as is evidenced by the fact that one company sold over 13,000,000 tons of ballast during the years 1913 to 1928.

In order to arrive at some definite information regarding the behavior of the several kinds of ballast in actual service, a questionnaire covering the chief factors was sent to 150 of the most important railroads in this country and Canada. Sixty replies were received, 35 of which contained sufficient data to be included in the tabulation. The replies used came from railways of the following classification: 16 lines of less than 1000 miles each, totaling 7738 miles; 11 lines of between 1000 and 5000 miles, totaling 27,783 miles; and 8 lines of over 5000 miles each, totaling 89,500 miles. The grand total of the mileage represented is 125,021 miles. The data have been summarized in Tables V and VI.

Most of the characteristics of different aggregates are so definitely observable from these tables that no further mention is necessary. Some of the less specific items covered by the survey but not tabulated, need brief statements.

Judging from the expressions of the engineers most concerned with ballast, the need of cleaning is brought about more by the introduction of foreign matter than by disintegration of the ballast in use. There is no general agreement as to the time nor the method of cleaning since special or local conditions control such matters. Some engineers find it cheaper to replace ballast than to clean it. Where replacements have been made, these have been of the same material in practically all cases according to the survey.

Opinions vary somewhat as to the effect of the aggregates upon ties and rails. Occasionally the question is raised whether an aggregate such as blast-furnace slag has an accelerating effect on the deterioration of rails or ties. In Table V it is answered negatively since nine railways pronounce its effect as "good" and three others as "fair." In Symposium No. 10 of the National Slag Association, the experience of engineers in all fields of construction answers negatively the question as to whether there is any corrosive quality in slag. More specifically applying to ballast is the work of Burchartz and Saenger[1] in which the corroding effect of slag was found to be no greater than that of most other samples of aggregate in which the metallic specimens were imbedded.

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[1] H. Burchartz and G. Saenger, *Archiv Eisenhüttenwesen*, Vol. 1, September, 1927.

A great deal of work has been done on the standardization of ballast by the Ballast Committee of the American Railway Engineering Association. Specifications have been outlined to cover weight, toughness, hardness, soundness, abrasion, cementation, cleanliness and the screen analysis of the materials chiefly used. The tests are made according to standard methods. The result of the committee's work has been a better understanding of the conditions imposed by service and an improvement in the quality and in the use of ballast.

According to the 1921 Manual of that Association the relative merits of the more widely used aggregates are in the following order: (1) trap rock or granite; (2) limestone or sandstone; (3) washed and crushed gravel; (4) blast-furnace slag; and (5) screened and pit-run gravel. A survey by the Ballast Committee of the National Crushed Stone Association in 1925 showed that 50 railroads aggregating over 110,000 miles of trackage indicated the same relative order of merit. On a numerical basis, assuming trap rock to be 100, the survey showed limestone or sandstone 87; washed and crushed gravel 78; blast-furnace slag 68; and screened and pit-run gravel 61. The amounts of the various materials needed annually for maintenance of a full ballasted track were found to be; trap or granite 3.3 per cent; limestone or sandstone 4.6 per cent; washed and crushed gravel 5.9 per cent; slag 7.9 per cent; and screened and pit-run gravel 9.9 per cent.

The size of the ballast is commonly 2½ in. maximum while the lower limit varies widely with the type of material. This permits a great variation in the behavior of the same ballast. It is to be expected that a more clearly defined minimum size will be forthcoming.

CONCLUSIONS

In dealing with a commodity such as ballast, economic considerations are paramount. Hence the availability of one type of material may preclude the use of another of better quality. Nearly all types of commercial aggregates show sufficient durability against weathering to fit them for use as ballast. The aggregate producers chief contribution to this field is the preparation of an aggregate which is properly sized, clean, and of sufficient mechanical strength to resist the stresses placed upon it.

Acknowledgment.—This opportunity is taken to express the appreciation of the author to the many railway engineers and state health officers for their courtesy in contributing data contained in this paper. Likewise to Messrs H. J. Love, Fred Hubbard, C. C. Hommon and George B. Gascoigne for the use of information contained in unpublished tabulations and reports.

NEEDED RESEARCH ON MINERAL AGGREGATES

By F. H. Jackson[1]

Synopsis

The paper discusses briefly a number of problems in the field of mineral aggregates which appear to the author to be of outstanding importance. Reference is also made to a number of points raised by the various contributors to the symposium in connection with needed research.

Attention is called to the fact that fundamental knowledge regarding the properties and uses of mineral aggregates has not kept pace with the actual use of such materials in construction. The need for a correlating agency to review and analyze existing test data as well as to coordinate future activities along this line is stressed.

Among the more important aggregate problems which appear to be in need of solution are the following:

1. Relation between character of aggregate and flexural strength of concrete.
2. Relation between character of aggregate and durability of concrete.
3. Standardization of laboratory tests for durability of aggregates.
4. Standardization of laboratory tests to indicate directly the quality of aggregates for concrete.
5. Relation between character of aggregates and stability of bituminous mixtures.
6. Standardization of a test for stability of bituminous mixtures.

In each of the papers entering into this symposium more or less consideration has been given naturally to the question of needed research in the particular field covered. Adequate treatment of the subject would be impossible otherwise. It seems desirable, however, to conclude the symposium with a short paper, summarizing the various comments which have been made along this line and in particular calling attention to what appear to be certain outstanding problems which it is believed should be emphasized.

Although, as defined tentatively by this Society,[2] aggregates comprise only those inert materials which when bound together by a matrix form concrete, mortar, plaster, etc., the symposium covers the subject in a somewhat broader sense by including consideration of materials for railroad ballast and filter beds. However, the major interest in so far as the Society is concerned has been in mortars, concrete and road materials and this review will accordingly deal principally with these uses of aggregates.

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[1] Senior Engineer of Tests, U. S. Bureau of Public Roads, Washington, D. C.

[2] *Proceedings*, Am. Soc. Testing **Mats.**, Vol. 28, Part I, p. 835 (1928); also 1928 Book of A.S.T.M. Tentative Standards, p. 251.

A reading of the several papers, each of which has been written from the point of view of an expert in his particular field, impresses one with the fact that our knowledge of the properties of mineral aggregates and the manner in which these properties affect the service behavior of structures has by no means kept pace with the actual use of such materials.

This is not particularly surprising when we consider that it has only been within the last twenty-five years or so that there has been any great demand for materials of this sort. Developments in the use of concrete and in road construction have been particularly rapid within this period, until at the present time these activities account for a very substantial part of the entire yearly consumption of mineral aggregates. At the beginning of this period, however, the engineer was faced with the necessity of selecting and using such materials without adequate information as to the service value of the various types and kinds of aggregates available. Under such circumstances it was inevitable that mistakes would be made both in the selection of materials and in their use. It takes time to secure, through research or experience, the necessary fundamental information regarding these matters, and construction could not be delayed. However, experience is a good teacher, especially if as a result the necessity of securing this fundamental information is brought home to the producer and user of the material and research thereby stimulated.

During the last few years a large and increasing volume of experimental data based on laboratory and field tests of aggregates has been accumulated. This work, in so far as portland-cement concrete is concerned, was instituted on a large scale in this country by the Portland Cement Association, which in 1916 established a research laboratory at Lewis Institute, Chicago. We are all familiar with the valuable work which has been accomplished by this organization as well as by the many agencies, both governmental and educational, which have been working on the problem of how to make better concrete. Many valuable reports have been published bearing on various phases of the subject.

In the field of bituminous road construction, we have the pioneering work of Richardson, Dow and others, extending back forty years or more. The theory of designing sheet-asphalt paving mixtures developed by Richardson was followed widely by engineers for many years. However, changing traffic conditions, as well as a desire on the part of certain investigators to study the economic phases of asphalt mixture design, has recently stimulated renewed interest in the subject, an interest which is reflected in this symposium in the papers by Hubbard and Skidmore.

Before going further into the subject of needed research in aggregates, the author would like to call attention to the need for some group or agency to review and digest the information which is available at the present time. Although a large number of reports and papers have been published dealing with the character and use of mineral aggregates, there has been a lamentable, though possibly unavoidable, lack of coordination in the work, which makes it extremely difficult to apply the data which have been accumulated. Furthermore, no systematic attempt has ever been made to review and analyze these data, not only in order that conclusions of practical value may be drawn but that needless duplication of effort may be avoided. The author believes that the decision of the Society to conduct a Symposium on Mineral Aggregates was a step in the right direction in that it will bring together and make readily available references to and comments upon most of the outstanding researches in the field of mineral aggregates. Moreover, these reviews have been made by authors who are recognized authorities on the subject.

Reference should also be made to the recent decision by Committee C-9 on Concrete and Concrete Aggregates to divide the work of the committee into a number of definite problems, assigned to individuals for study and report. It is believed that this plan, by definitely fixing responsibility and thereby stimulating interest on the part of individual members, will result in clearing up many debatable points regarding the properties and uses of concrete aggregates much sooner than would be possible otherwise. This plan might well be tried out by other committees.

What are the major problems in the field of mineral aggregates now calling for solution? What properties of aggregates are in need of study and how may they be determined? What tests are in need of standardization and what tests need to be correlated to service behavior in order that the results may be properly interpreted? In the following discussion an attempt will be made to answer some of these questions.

AGGREGATES FOR PORTLAND-CEMENT CONCRETE

The two major structural requirements for portland-cement concrete are strength and durability. In addition, concrete for special purposes must possess special characteristics: such as resistance to wear in the case of concrete pavements, floors, etc.; resistance to chemical agencies, as in the case of concrete exposed to sea water and alkali; and resistance to extremely high temperatures, such as concrete exposed to the fire hazard.

Strength:

Strength is an essential characteristic of all concrete and much study has been given to the various factors that affect it. These studies have so far been largely confined to compressive strength, due to the fact that, for most types of concrete construction, resistance to crushing is the property in which we are principally interested. They have shown, in general, that the effect of various aggregate characteristics other than gradation are of relatively minor importance so long as the aggregate is structurally sound and free from deleterious materials.

However, in the case of concrete pavements, we have to deal with flexural as well as compressive strength, and recent researches with coarse aggregates have shown that such characteristics as surface texture, angularity, absorption, etc., are of considerably more significance when studying flexural strength than when compressive strength alone is considered. Studies of fine aggregate to determine to just what extent similar characteristics affect the flexural strength of concrete should be made as well as studies to develop standard methods of making flexure tests of concrete. Further researches are also needed in order that a practical method of designing concrete mixtures to meet certain flexural strength requirements may be devised.

These are outstanding problems and their solution should enable us to utilize to much greater advantage the available aggregate for any concrete work in which flexural strength is a factor to be considered.

Durability:

The effect of type of aggregate upon the durability or resistance to weathering of concrete is a subject of very great interest and offers a fertile field of research. Progress in determining the factors affecting durability has been slow, due both to the difficulty of making satisfactory accelerated weathering tests in the laboratory and to the difficulty of properly interpreting the causes of failures in structures. In an effort to throw light on this subject, attempts have been made within recent years to conduct freezing-and-thawing tests of concrete in the laboratory. The tests so far made have been of value principally in helping us standardize methods of conducting such tests rather than for any real information as to the effect of the kind of aggregates upon resistance to frost action. It is hoped that as our knowledge of how such tests should be made increases, a standard method of procedure will be developed which can be used in making a thorough study of durability and the factors which affect it.

The desirability of knowing something about the durability of aggregates in advance of actual use resulted a few years ago in the adoption for laboratory use of the so-called sodium-sulfate test for soundness. This test was first used by Brard in connection with investigations of the soundness of building stones. Recent attempts to apply the results of this test to concrete aggregates have indicated that the detailed laboratory procedure will have to be specified much more exactly if concordant results are to be secured. There is also necessity for a correlation study of the results of this test on aggregates of different types and characteristics, both with actual freezing and thawing tests on concrete in which the aggregates are used, and observations of the service behavior of the concrete structures themselves.

Other Properties of Aggregates Which Affect Their Value for Use in Concrete:

There are many debatable points as to the effect of various characteristics of mineral aggregates which are of interest to both producer and user and which should be cleared up either by original research or by a thorough study of existing data which may bear upon the matter in hand. Specifications for mineral aggregates under consideration both by this Society and the American Concrete Institute contain provisions for limiting quantitatively the maximum permissible percentages of various deleterious substances or materials with certain undesirable characteristics. However, existing data indicating just what these limits shall be are very meager. For instance, what is the maximum limit for shale in sand for concrete pavement? There is information to indicate that more than 1 per cent of shale in sand may be permitted under certain circumstances. How may flat and elongated pieces be defined and what is the maximum quantity permitted under various conditions of use? To what extent may the dust of fracture as a coating on crushed stone be permitted? What should be the maximum loss by elutriation in sands for use in concrete pavements, and is the present silt test a satisfactory method of determining the amount of undesirable fine material? What requirements are necessary to write into specifications for "slag sand" and "stone sand" in order that these materials may be used satisfactorily as fine aggregate? These are just a few of the questions affecting the production and use of mineral aggregate which have not been satisfactorily answered, in spite of the fact that many isolated and unrelated investigations have been pursued from time to time in an effort to throw light upon these matters. Here again we see the need for a correlating research agency to direct and coordinate these efforts.

Methods of Testing Aggregates:

There are two time-honored tests for quality of concrete aggregates which at the present time are causing the testing engineer considerable trouble: The "Deval abrasion" test and various modifications thereof as applied to coarse aggregates, and the so-called "strength-ratio test" for concrete sand. Neither test is a direct measure of the concrete-making qualities of the aggregate. They both give information which if properly interpreted will be of value; however, attempts to apply the results of these tests by engineers who are not familar with all of the factors involved frequently result in absurd decisions, thus bringing the test methods themselves into disrepute. This is particularly true of the strength-ratio test which, due to the incidental effect of gradation, gives results which cannot be interpreted directly. In spite of these facts, engineers have been loath to abandon these tests because, when properly interpreted, they do yield valuable information. Research is needed to develop tests which will give directly the essential qualities of aggregates. At the present time it would seem as though a test of the mortar or concrete in which the aggregate is to be used is the best solution because we are after all only interested in the physical properties of aggregates to the extent that they influence the performance of the mixture. The principal objection to rating aggregates on this basis is the time and expense involved in making concrete specimens, especially for control work.

Effect of Aggregate on Volume Change and Fatigue of Concrete:

The factors affecting volume changes in concrete due either to temperature or moisture as well as those affecting the resistance of concrete to repeated loads (fatigue) are of great interest, especially in the case of pavement concrete. Just what part the quality of the aggregate plays in influencing these properties is a question. Very few research data are available on either of these points and information is needed.

Fire Resistance:

A review of Ingberg's paper in this symposium indicates that, while coarse aggregates have been classified definitely with respect to relative fire resistance, there is need for information as to the effect of fine aggregates of different mineral composition, impurities in aggregate, etc. There is also need for a standard test for fire resistance that will classify concrete as to fire resistance without reference to the type of structural member in which it is applied.

Gradation:

There has probably been more work done on the effect of size and grading of concrete aggregates than on any other characteristics. It has been shown that, so far at least as strength is concerned, gradation is chiefly important because of its influence upon the quantity of mixing water required which in turn controls the amount of cement and therefore to a large extent the cost of producing concrete to a definite strength requirement. The fundamental water-cement ratio - strength relation has been well established. There is need, however, for further work to determine whether or not the water-cement ratio governs durability as well as strength—in other words, whether the gradation of the aggregate influences durability solely because of its effect on the amount of water in the cement paste. We need also a satisfactory measure of workability. It is not sufficient to judge workability by some arbitrary standard set up in the laboratory. Workability is a relative term and whether or not a concrete is workable depends as much on the equipment and methods used in handling and placing it as upon any other factor. Viewed from this standpoint, the only satisfactory measure of workability is the result secured on the job in terms of the cost of securing this result. The effect of character and gradation of aggregates on workability as here defined deserves study.

AGGREGATES IN BITUMINOUS ROAD CONSTRUCTION

Many of the comments which have been made in connection with the use of aggregates in cement concrete apply with equal force to aggregates used with bituminous binders. Here again, our principal studies in the past have had to do primarily with size and gradation rather than with other aggregate characteristics such as angularity, surface texture, porosity, etc. In the case of bituminous mixtures, efforts have been confined largely to the design of mixtures for greatest density, and as both the final density of the pavement and the theoretical density of the mixture are functions of the grading of the mineral aggregate the importance of studying this property is apparent.

The papers by both Hubbard and Skidmore in the symposium emphasize the necessity of designing bituminous mixtures which will remain stable, that is, resist deformation under traffic. The various factors which influence stability are discussed with special reference, of course, to the effect of aggregates. Of the several recently proposed methods for determining stability, two are described in these papers. Other methods which have been proposed from time to time include an impact test by Besson, a rod penetration test by McNaughton, a ball penetration by Howe, static deformation load test by Ulman and Milburn, and the squeeze test by Emmons and Anderton.[1]

There is also being developed at this time at the U. S. Bureau of Public Roads an apparatus for determining stability by measuring the resistance of a specimen to horizontal deformation induced by a succession of rolling loads designed to simulate the action of traffic.

It appears to the author that the outstanding need in this field is the standardization of a mutually satisfactory stability test which can be used by all of those interested in studying the behavior of bituminous mixtures. The closer such a test approximates the action of traffic the better. Until such a test is agreed upon it does not seem that much progress will be made in studying mixture performance unless we know that the various devices which are being used all measure that essential property in the material which makes for stability in the pavement. Information is needed also as to the effect of various physical properties of aggregates including mineral fillers, such as surface texture, porosity, shape of grain, etc., on the service behavior of bituminous mixtures. Here again a recognized laboratory test on the mixture, properly correlated with service behavior, would be of great assistance.

There is also need for more definite information as to the significance of the various specification tests now employed to control the quality of aggregates used in bituminous mixtures. This is especially true of coarse aggregates, such as crushed stone, gravel and blast-furnace slag used in the bituminous macadam types. Are we properly interpreting the results of the Deval abrasion test—that time-honored measure of aggregate quality? What relation exists between this test and the consistency of the bituminous binder to be used? What relation exists between the absorption of aggregates and the amount and consistency of the bituminous binder? How should the results of modified Deval abrasion tests on gravel or slag be interpreted in order to insure a quality comparable to crushed stone? These problems are all of great interest to both producer and user and their solution should go far towards a more intelligent use of such materials in road construction.

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[1] From "Researches on Asphaltic Paving Mixtures," by Prévost Hubbard and F. C. Fields, *Proceedings*, Thirty-first Annual Convention, American Society for Municipal Improvements, Des Moines, Iowa, 1925.

AGGREGATES AS FILTER BED MEDIA

Kriege's discussion of the properties and uses of materials for filter beds leaves little to be added. The greatest emphasis is placed properly upon durability. This is undoubtedly the most important characteristic of aggregates used in such work, due to the severe weathering conditions imposed. Additional investigations are needed more definitely to correlate service behavior with accelerated soundness tests, particularly the sodium sulfate test. Here again we have evidence as to the necessity for securing information regarding the durability of aggregates. It is the one outstanding problem and its solution will be of interest to all users of aggregates in structures exposed to the weather. We must have durable aggregates. On the other hand, we cannot afford, for economic reasons, to rule out materials simply because they fail to meet some arbitrarily chosen test for soundness until it can be shown that such test actually does measure the resistance of the material to natural weathering.

The next most important characteristics of filter bed material appear to be size and cleanliness. As Kriege points out, these properties can be controlled by the producer and our interest lies principally in knowing what requirements to specify, what tolerances to allow, etc. These questions may best be answered by studying actual installation where information is available regarding construction materials and method and noting the results secured.

In conclusion, the author wishes again to voice the hope that this symposium will stimulate a renewed interest in the study of the properties and uses of mineral aggregates. He wishes also to emphasize again the importance of correlating our activities along this line so that the results of accomplished as well as proposed research may be made more readily available.

DISCUSSION ON MINERAL AGGREGATES

INSPECTION OF AGGREGATES

MR. M. H. ULMAN.[1]—Mr. Crum discussed tolerances in specifications. This subject needs considerable clarification, for specifications should be so written as to include the tolerances which will be permitted and no variation should be allowed from them. Specifications should be written by one who is thoroughly familiar with the desirable qualifications of materials for the intended use. If variations over and above the specification tolerances are permitted by the consumer it is probable that this will become progressive as additional requests are received from the producing interests when materials fail to comply with the specified requirements.

In the papers presented, no reference was made to the desirability of the producer having his own testing organization, so that he can be assured that all materials shipped comply with the requirements of the particular consumer, and by this method, which I feel is the most desirable, the final control can be exercised by the consumer of the materials as received.

Mr. Rea has discussed a number of factors to be considered in inspection. To these I would add the amount of coatings on fragments of rock and any clay lumps in the shipment. This information together with information on amount of foreign material is of extreme importance when any controversies occur between the producer and consumer as to the materials' compliance with specifications. If condemnation has been made for any of these causes based on visual inspection it is most difficult to satisfy the producer that the correct interpretation has been made.

MR. R. W. CRUM.[2]—I think in most cases it is possible to set the tolerance limits in the specifications, and whenever possible they should be included therein, and not left to the discretion of the engineer or inspector on the work.

MR. A. S. REA.[3]—In regard to the determination of coating on aggregates referred to by Mr. Ulman, I feel personally that this can best be determined in the field by visual examination. We have found that in many cases it cannot be determined satisfactorily on samples sent to the laboratory, and I believe, too, that a quantitative determination in the field may not in every case be conclusive. For example, a very small percentage of adhesive clay may be more objectionable than a much higher percentage of a more granular coating. Our instructions to engineers and inspectors provide that all aggregate for concrete shall be examined for coating on the job.

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[1] Assistant Engineer of Materials, Pennsylvania State Highway Dept., Harrisburg, Pa.

[2] Director, Highway Research Board, National Research Council, Washington, D. C.

[3] Chief Engineer, Bureau of Tests, Ohio Department of Highways, Columbus, Ohio.

MR. L. E. WILLIAMS.[1]—With reference to producers having their own plant inspection in order to produce material which will ultimately be acceptable on destination inspection, we have found that seldom will two men take separate samples of a carload of material and get identical results. If the material is anywhere near the limits of the specifications, we have to rest our case on the inspection of the people to whom we are furnishing it. If that inspection is made at the plant, that ends the argument; rejected cars can be taken care of, whereas rejection at destination often involves very considerable loss to the producer both in freight charges and also ultimate disposal of the material. Your own forces might make the most conscientious inspection possible at the plant and yet, with perfectly conscientious work on the part of the purchasers, by a difference in sampling, the car might be rejected.

In our own state we have exceptionally close cooperation, at least so far as the state highway department is concerned. It is practically all plant inspection, and the spirit of those inspectors almost invariably is to assist the producer to get the material on the job that is acceptable; with that cooperation we have comparatively few rejections, providing the producers do what they should do. On the other hand, we have other customers where we have destination inspection on almost identical specifications, and, due to differences in sampling at destination, we often have considerable trouble.

There is one other point that I should like to touch on; it is not strictly a matter of sampling, but it is related to the ultimate use of aggregates. Some few years ago, our company in cooperation with the University of Michigan, had two men, who were post-graduates at the University, carry out investigations of sampling methods with particular reference to how the material carried through to destination. By that I mean taking a series of carloads at the plant, taking samples by several different methods, then following those carloads through to destination, taking samples out of the stock piles where used, out of bins where bins were used, and ultimately taking batch samples at the mixer. Incidentally, we developed a method of taking samples off the loading belt, where a loading belt was used at plant, that we believed gave the most representative possible sample of the car in question. At the mixers, in a great many cases, the material was no longer recognizable, due primarily to segregation in handling.

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[1] Plant Manager, Ray Sand and Gravel Co., Detroit, Mich.

Speaking at least for Michigan producers, we are perfectly willing to furnish materials to the best of our ability under any reasonable specifications—I mean specifications written sufficiently broad to be within the limits of normal plant production. However, we feel that we should not be required to meet exacting specifications when that same material is handled in such a way that, by the time it reaches its destination, due to segregation and methods of handling, one gets one batch which may run outside of the specifications on one side and the next batch may run outside in the other direction. We are not asking for leniency in specifications or in inspection, but in the interest of better concrete and materials being used to the best advantage, we do feel that there are a lot of weak points in the handling of materials between the production plant and destination, that should receive careful attention.

MR. A. T. GOLDBECK[1] (*presented in written form*).—It can be safely said that no other part of the procedure concerned with the purchase of aggregates gives quite so much trouble to both producer and consumer as the question of adequate inspection. Both parties have their own special views on this question and the methods pursued are oftentimes admittedly not the best, but are governed by expediency. The commercial producer would like to feel that when carloads of aggregates leave his plant they are entirely suitable for the work and will not be rejected when they arrive at their destination. Unfortunately, however, with the method of inspection practiced in many of the states, that of inspecting at point of delivery, it too frequently happens that many carloads in a single shipment are rejected for some reason which sometimes is trivial and could readily have been obviated had the matter been thoroughly understood at the producing plant.

In some localities high-class materials are not available and it becomes necessary to use the best materials procurable which are satisfactory, provided they are rigidly controlled. When materials such as these are inspected at the site of the work, they are sometimes rejected and the producer suffers a considerable loss with the result that certain producers have refused to ship material unless plant inspected. At other plants producing unquestionably high-grade material, with methods of production which in every way are first-class, plant inspection becomes a needless expense to the State for it is seldom the case that such material does not comply with specifications.

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[1] Director, Bureau of Engineering, National Crushed Stone Assn., Washington, D. C.

Too commonly inspectors at the work, instead of following specifications, have their own ideas as to how the materials should be graded and the same plant shipping aggregate to different inspectors in the same state has frequently had to change the gradation merely to suit the individual whims of the different inspectors. Yet all of this aggregate was supposed to comply with the same specification. Such conditions, obviously, should not exist. This is a matter of educating the inspectors by the State.

Too commonly expensive rejections are made on the basis of the most superficial "toe" inspection by inspectors who have no knowledge of production methods and their effect on gradation. Some inspectors feel it their duty to reject a given percentage of shipments to demonstrate their efficiency, completely overlooking the fact that their rejections lead to high costs and that it is really their duty to assist the producer in every possible way so that his material will be accepted. Complete and sympathic understanding between producer and the inspection force will do much toward the production of better materials, thus speeding up the work and leading to better construction. Good engineering and good inspection demand not that the materials supplied to the work be perfect in every detail irrespective of the cost involved in their production but rather that they be of a quality entirely satisfactory for the work and produced at a reasonable cost.

MR. SHREVE CLARK.[1]—Mr. Goldbeck requests that all the provisions for tolerances be written into the specifications. That would be very good if it were possible, but when we encounter coatings on stone, I know of no way in which that material or its condition or characteristics can be described so that an inspector will be able to accept or reject it on that factor.

On many types of crushed stone varying layers of dust are collected, dependent upon the moisture in the air, whether it is foggy or rainy weather, and the dust conditions about the quarry. This dust may or may not be of such a character as would interfere with its use. In many cases the layer of dust does not interfere with the use use of this material as a concrete aggregate. However, in case it is to be used with bituminous material, it is absolutely fatal, and I know of cases where the layer of material on the outside of granite or sandstone is not at all detrimental for use as concrete aggregate, but it will prevent the adhesion of bituminous material.

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[1] Senior Assistant Engineer, Tests, Virginia Department of Highways, Richmond, Va.

I have a case in mind of a limestone that had a slight layer of dust on it, or rather you might say slime, because it was a very slight layer indeed, but the material was accepted because it was considered that stone of this character was satisfactory for concrete. It happened to be mixed in a mixer with a cutback asphalt for the purpose of putting it as a surface on a road. It was well coated in the stock pile, but in a day or so a very heavy rain fell and completely washed the bituminous material off the surface of the pile and the whole thing was a complete loss, all on account of a surface layer of fine material on the stone.

If Mr. Goldbeck will tell us how to write specifications to prevent such a condition, we will be glad to know it, because that holds for dusty layers on the surface of any aggregate.

MR. D. D. MCGUIRE.[1]—I think that we, as an organization, should have respect for inspection, in that we should guard against such clauses in specifications as "clean," "sound," "durable," and "free from." One cannot ask an inspector to adhere to specifications that contain clauses impossible to produce. I think this Society should consider this very carefully in developing specifications.

MR. STANTON WALKER[2] (*by letter*).—Messrs. Crum and Rea have confined their discussions to inspection methods. While this subject is of utmost importance to the producer and consumer of mineral aggregates, it is felt that a thorough analysis of it requires more complete consideration of the principal basis of difficulties and misunderstandings resulting from inspection—the specifications. Inadequate and unscientific specifications probably are responsible for more of the difficulties laid at the door of inspection than any other cause. Another source of misunderstanding is the interpretative clause, not incorporated in the specifications but enforced as a part of them, which is attributable to the inadequacy of the specification.

Inadequate specifications, for the most part, are not due to carelessness or lack of thoroughness on the part of the engineer writing them, but are due to lack of definite information on the effects of the various characteristics of mineral aggregates on the finished product. The familiar and indefinite clause "free from injurious amounts of" is the bone of contention in many instances. Specifications for grading written without sufficient regard to the characteristics of available materials and which cause the producer to work within a narrow range near the limits of the specifications are frequently the cause of the inspector having difficulty in deciding on the suitability of the material.

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[1] Engineer of Tests, Tennessee Highway Dept., Nashville, Tenn.

[2] Director, Engineering and Research Division, National Sand and Gravel Assn., Washington, D. C.

Many of the difficulties arise, of course, from the causes emphasized by Messrs. Crum and Rea—faulty or unintelligent inspection methods. These, however, can be corrected by education. No one is more anxious than the inspector to have a definite, clear-cut basis for the determination of the suitability of a material.

Examples of misunderstandings and hardships resulting from the several causes mentioned above could be given without end. While a complete discussion of them might leave the erroneous impression that the life of the mineral aggregate producer is almost entirely filled with difficulties with inspectors and specifications, a few examples should be of interest.

Many specifications for grading leave entirely too much to the judgment of the inspector or to supplementary clauses not a part of the specifications. The specifications for grading of gravel of a prominent state highway department requires only that 100 per cent shall be finer than $2\frac{3}{4}$ in. and that not more than 5 per cent shall be finer than $\frac{3}{8}$ in. Except for the phrase "graded uniformly," a literal interpretation of such a specification permits the use of, for example, 2 in. to $2\frac{3}{4}$ in. material, or $\frac{3}{8}$ in. to $\frac{1}{2}$ in. material. Obviously, such a range in sizes is not permitted and personal opinion must enter into the determination of the suitability of the gravel.

Specification limits on soft particles, with the lack of definite information as to how to identify a "border-line" soft particle, are a frequent cause for lack of consistency in accepting or rejecting material by different inspectors.

To cite the examples of unintelligent inspection would require too much space and serve no definite purpose. That difficulty is not fundamental and will be corrected with education and increased knowledge. Greater emphasis on clear-cut, definite, and scientific specifications will make inspection positive and permit the producer to furnish material which will make inspection a matter of routine.

FINE AGGREGATES IN CONCRETE, MORTAR AND PLASTER

MR. W. F. PURRINGTON.[1]—Referring to page 760 of Mr. Gonnerman's paper, he says: "In general, natural sands consist of rounded particles." I wish to ask whether he means this statement made in that way? I was very much surprised to read this because in our state I hardly know what a rounded particle of sand looks like. We find that our sands are angular. I wonder if the statement has reference to all sands?

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[1] Chemist and Testing Engineer, New Hampshire Highway Dept., Concord, N. H.

MR. H. F. GONNERMAN.[1]—What I meant to imply by the statement to which Mr. Purrington refers was that most natural sands consist of particles having rounded rather than sharp, angular edges as is the case with crushed stone sands. There are sands that do not consist of spherical particles as pointed out by Mr. Purrington. Natural sands, however, since they have been formed by wave action or weathering do, in general, consist of particles having rounded edges, although that may not always be the case.

MR. GOLDBECK (*presented in written form*).—Mr. Gonnerman has presented the subject of fine aggregates in a very interesting way. There are certain points in his discussion upon which comment might well be made. He states that the shape of particle has little effect on concrete strength. In the past we have always thought of strength of concrete in terms of compression and no doubt his statement applies to compressive strength, but there is reasonable doubt as to whether such a statement is equally applicable to the case of tensile or beam strength.

He has stated that in hydraulic structures deterioration takes place at the water line. It would seem that this statement might well be followed up by the statement that such action also takes place even when the aggregates are sound. Thus, disintegration very frequently is due to defects in the concrete other than in the fine and coarse aggregates.

Speaking of screenings and their use as a fine aggregate in concrete, mention is made of the fact that they require more water to create a given consistency and that, therefore, when the concrete is designed by the water-cement ratio method, more cement is necessary. The question may well be asked, why should a given water-cement ratio be specified and made applicable irrespective of the kind of aggregates used. The water-cement ratio law applies strictly, only when the same aggregates are employed. It is an established fact that a higher water-cement ratio may be used with certain aggregates than with others with no difference in resulting strength.

It is stated that dust in screenings is harmful because, when too much mixing water is used, the dust and other fine material rise to the surface where they form a weak, friable layer called laitance and produces a concrete which is unable to resist wear or weather. Attention is called to the fact that laitance may be formed even if all fine particles are absent except those in the portland cement. It is well known that excessive water will render portland cement non-cementitious and fine particles of cement which come to the surface in the presence of excess water will then act as so much inert material. When the water rises to the surface of a concrete road in finishing, an exceedingly high water-cement ratio is created. This causes two effects: (1) a considerable weakening of the surface layer, and (2) creation of a highly porous surface layer.

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[1] Manager, Research Laboratory, Portland Cement Assn., Chicago, Ill.

Both of these effects make the action of frost very destructive. This probably offers an explanation of the phenomenon of scaling. Scaling practically never occurs in the absence of frost.

The effect of stone dust has been investigated by the writer to some extent both with regard to its effect on the strength of concrete and on the wearing quality of concrete. Some of the results obtained are included in an appendix to the current report of Committee C-9 on Concrete and Concrete Aggregates.[1] They show that small percentages of stone dust, such as may occur on crushed stone produced in wet weather really have a negligible effect.

MR. WALKER (*by letter*).—Mr. Pearson, very properly, has pointed out our lack of knowledge concerning the function of the fine aggregate in controlling the quality of mortar and plaster. The writer is not prepared to agree, however, that, because we do not have definite information on the effects of the various characteristics, we are justified in accepting his conclusion "that almost any materials worthy the name of fine aggregate can be used satisfactorily as aggregate for ordinary mortar and plaster."

Mr. Pearson emphasizes the importance of plasticity and volume change and points out the relative unimportance of strength. At the same time he recommends a basis for determining the suitability of sands (the plastic mortar test) which, in effect, is a method for arriving at the proportions of aggregate to cementitious material which will produce approximately the same strength. By the use of this test the acceptance of a sand depends (within limits) on the cost of the mixture which will produce a suitable plasticity, workability, and strength with a given water-cement ratio, without regard to its grading or other characteristics.

It is known that the richness of the mix and the characteristics of the aggregate influence the plasticity and volume changes of mortar. These influences are independent, or certainly not proportional to, the strength. While no exact information is available on the effects of these factors, it is known that they are important and something is known concerning the trends of their effects. Experience and certain tests indicate that coarser gradings, the absence of extremely fine sizes and leaner mixtures tend to produce lower volume changes. In general, therefore, the factors which produce more favorable conditions as to volume changes make for less favorable conditions as to workability. It will be seen also that variations in mix which decrease volume changes also decrease strength and that, generally, variations in grading which decrease volume changes increase the strength.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part I, p. 301 (1929).

On the basis of this evidence it would appear that specifications written in the light of complete information would represent a compromise among workability, volume change, and strength. While present specifications, as Mr. Pearson points out, are based on little more than opinion, that opinion is formed by experience, and it is believed by the writer that it takes the factors named above into account in some degree. The writer does not believe that the requirements of existing specifications, as inadequate and unsupported as they are, should be scrapped in favor of the belief "that almost any materials worthy the name of fine aggregate can be used satisfactorily as aggregate for ordinary mortar and plaster."

MR. W. H. HERSCHEL[1] (*by letter*).—In the paper by Mr. Pearson it is stated that "In regard to plasticity . . . if we accept the definition now tentatively before the Society we may declare that there is no recognized method nor present available means for determining the 'complete force-flow relation' in mortars and plasters." While it is believed that the main difficulty is that available means are not used or modified as needed for concrete and mortar, I wish here to consider nomenclature rather than apparatus.

The reference to "complete force-flow relation" is to the proposed definition for consistency: "consistency . . . is defined by the complete force-flow relation," and this is also true of plasticity, which is one variety of consistency. Pearson could not well use the word consistency in this connection since he indicates that consistency can be measured by the flow table. Jackson, on page 860 of the same Symposium on Mineral Aggregates, speaks of the consistency of bituminous binder, and this use of the term is in agreement with the proposed definition and is contrary to the tendency to regard consistency as depending only upon the water content.

It is believed that the best way out of this confusion in regard to the use of the word consistency is to adopt the proposed definition. Since with plastic materials, such as mortar and plaster, the force-flow relation cannot be represented by a straight line passing through the origin, it follows that the consistency of a plastic material cannot be defined by any single numerical value whatever, no matter by what method such value may be obtained. It must be admitted, however, that as with the penetration test for greases and petrolatum,[1] consistency may be *adequately* defined, in some cases and for some purposes, by a single value, even though not *completely* defined.

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[1] Associate Physicist, U. S. Bureau of Standards, Washington, D. C

When water is considered to be the only material which can influence "consistency," resort must be had to the word "workability" to indicate the effect of adding other materials. The right amount of oil, for example, must be added to whiting to make putty, and this is just as much a question of obtaining a suitable consistency, that is, a suitable force-flow relation, as when water or some fine aggregate is added to mortar.

In order, however, that a material may be suitable to work or spread, it must remain of suitable consistency from the time the work is begun until it is finished. This involves a new idea not represented by the force-flow relation, namely, the change in consistency with time. The term "workability" is thus needed to refer to the property of being "readily manageable during the operation in question," in accordance with Bingham's definition.[2] The term is not needed to refer to the effect of admixtures of various materials upon the consistency.

MR. J. C. PEARSON[3] (*author's closure by letter*).—Mr. Walker's comment on the author's conclusion that "almost any materials worthy the name of fine aggregate can be used satisfactorily as aggregate for mortar and plaster" is the natural reaction of a conservative engineer to a statement which is too general to mean much of anything, but was designed to serve as a friendly stimulus to those who are most likely to be concerned with the establishment of suitable and reasonable limitations on fine aggregate for mortar and plaster. The statement is based on the fact that such materials are being used everywhere for this purpose, and no very positive evidence has been presented to show that they are unsuitable. The author is aware that experience with gypsum plaster furnishes some basis for the preference for a coarser sand, but it has not been conclusively shown that the sacrifice in plasticity or workability involved is attended with the maximum of economy in arriving at the desired result.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 930 (1927).

[2] E. C. Bingham, "Some Fundamental Definitions of Rheology," see p. 909.

[3] Assistant to Chemical Engineer, Lehigh Portland Cement Co., Allentown, Pa.

Mr. Walker's argument against the plastic mortar test for strength is fair enough. The presentation in considerable detail of the interesting strength relations obtained was designed to point out the general usefulness of the test, and was not intended to convey the thought that strength helps particularly in the selection of a fine aggregate for mortar or plaster. In fact the *only* thing we get from the strength determination is a warning signal to examine a little more closely a fine aggregate that is badly out of line in the lower bracket. The chief value of the plastic mortar test in its application to the subject of the paper is to indicate whether enough of the fine aggregate under test can be used to provide an economical mixture, and at the same time give desirable workability.

Mr. Walker's concluding remarks about a compromise among workability, volume change and strength, are again the natural attitude of one who bases his inferences mainly on the behavior of cement mixtures. If we subordinate the strength requirement, as the author has argued, we are really discussing a compromise between workability and volume change, and since we know almost nothing about volume change in mortars and plasters applied to absorptive bases, our compromise must favor workability until we have more conclusive evidence against a type of fine aggregate that contributes to this desirable property. Let us assume that the type of aggregate we have in mind is one that contains a larger proportion of material passing the No. 50 sieve than is permissible under Specifications C 35 - 25. Let us assume that somewhat more water will be required to gage a mortar containing such sand than one containing a coarser sand. This additional water represents additional potential void space in the set mortar, and therein lies the potential capacity for greater shrinkage. But the absorptive base removes much of this water and brings about a densification, which in the case of plaster is further promoted by the "tightening down" process—a compaction brought about by retroweling or refloating. It may be argued that the more finely graded mixture still holds more water, after it is in place, than the coarser, but now we are confronted by the advantage of the finer mixture in its greater workability, and regardless of its effect in helping to reduce voids to a minimum, we now have to weigh the disadvantage of residual shrinkage against the positive advantage of higher workability. As stated repeatedly in the paper, we do not have the necessary data for accurately weighing these advantages and disadvantages, but it is the author's opinion that improved workability, within reasonable limits, will tend to give a better and more economical finished job.

The author particularly welcomes Mr. Herschel's discussion, because, although brief, the thorough digestion of it by all those interested in plasticity or workability of concrete, mortar or plaster, will help greatly to clarify the haze that prevents some of our most prominent concrete investigators from seeing clearly the distinction between such properties as flow or slump and consistency or plasticity.

The point in Mr. Herschel's second paragraph is very well taken, and the author is happy to acknowledge the correctness of it. The only excuse for using the word "consistency" was that the meaning would be clearer to most readers, from long familiarity with the word as commonly used. It would be well, perhaps, to advocate the use of the term "wetness" in place of the term "consistency" as now frequently used in the specifications and literature. While "wetness" may be just as inaccurate or objectionable as "consistency," at least it has the advantage of constantly reminding us that we adjust our flows or slumps or Vicat needle penetrations by change in the water content of our mixtures, and that we do not thereby determine the consistencies, that is, the complete force-flow relations, of the mixtures.

AGGREGATES IN BITUMINOUS MIXTURES

THE CHAIRMAN (*Mr. H. S. Mattimore*[1]).—There is one question of information I would like to speak of. Mr. Skidmore is not present, but perhaps someone else can supply the information desired. I wonder at the statement that a pressure of 5000 lb. is equivalent to rolling with a ten-ton roller.

MR. PRÉVOST HUBBARD.[2]—Mr. Skidmore uses direct compression of 5000 lb. with practically no kneading action, which, to his mind and according to his experience, has given about the same degree of compaction for the mixes he has tested as he gets with rolling under the conditions he has stated. This does not check exactly with our own experience in that connection. We use a maximum of about 3000 lb., but we tamp our mixture before we apply the direct compression, and we get practically the same degree of compaction. Our objection to such a high direct loading is that in a good many sands it causes breaking down of the particles, and we try to avoid this as much as possible.

MR. ULMAN.—Mr. Hubbard, in presenting his paper, discussed the effect of variation in filler on the stability value of the mixture. To what extent is the marked variation mentioned due to variation in the specific gravity of the fillers?

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[1] Engineer of Materials, Pennsylvania State Highway Dept., Harrisburg, Pa.

[2] Chemical Engineer, The Asphalt Assn., New York City.

MR. HUBBARD.—There was very little difference in the specific gravities of these fillers. There was, however, considerable difference in their degree of fineness.

MR. BERT MYERS.[1]—I should like to ask Mr. Hubbard how he determines the quantity of asphalt that a sand will adsorb.

MR. HUBBARD.—The method that we use in determining the adsorption value of two sands is to take individual fractions of the two sands and also the total sand product. Weighed portions of the sands are well shaken in a standard solution of asphalt in carbon disulfide or benzol. We then allow the sand to settle, and calibrate the color of the supernatant liquid against the original solution by titrating or diluting with the original solvent. From the amount of dilution required it is easy to calculate the weight of asphalt which has been adsorbed by a given weight of sand.

MR. GOLDBECK (*presented in written form*).—In connection with the effect of commercial aggregates on stability of bituminous mixtures, much work remains to be done on the matter of fillers. Our specifications very inadequately describe the physical characteristics of mineral fillers and it is suggested that work might well be done on the effect of the filler on the weather resistance of the bituminous material, for there are indications that certain types of fillers may have a detrimental effect in the presence of water.

MR. H. W. SKIDMORE[2] (*author's closure by letter*).—In regard to the matter of 5000 lb. per sq. in. molding pressure (vertical), we have found crushing of mineral aggregate under this pressure very rare, indeed. It has occurred only in the case of extremely soft aggregate and even then in no greater degree than obtained in the actual pavement. The degree of compression obtained by this method of molding test cylinders is usually almost exactly that secured in the field by means of 10 to 12-ton rollers of the 3-wheeled type, which are almost universally employed.

Tamping is entirely foreign to field compression under rollers; it is difficult to standardize and introduces a compression factor incapable of accurate measurement. Direct vertical compression is admittedly the ideal type for bituminous paving mixtures. There is a strong feeling in the industry that rolling is a very inefficient means of compression, and inventive efforts are being directed toward the practical adaptation of vertical direct compression to the requirements of paving construction.

A direct pressure of 3000 lb. per sq. in., alone, will not duplicate field densities, and when supplemented with tamping, frequently, if not usually, exceeds them. The compression afforded by slowly moving 3-wheeled rollers is mainly vertical pressure and in no sense tamping. Obviously it is very difficult to simulate exactly roller action in the laboratory. We feel that the simplicity and ease of accurate measurement of our method together with its economy of time and its similarity to the main compressive force of rolling (as now done in the field) establish it as the most logical means of compression in the laboratory.

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[1] Engineer of Materials and Tests, Iowa State Highway Commission, Ames, Iowa.

[2] Director, Chicago Paving Laboratory, Chicago, Ill.

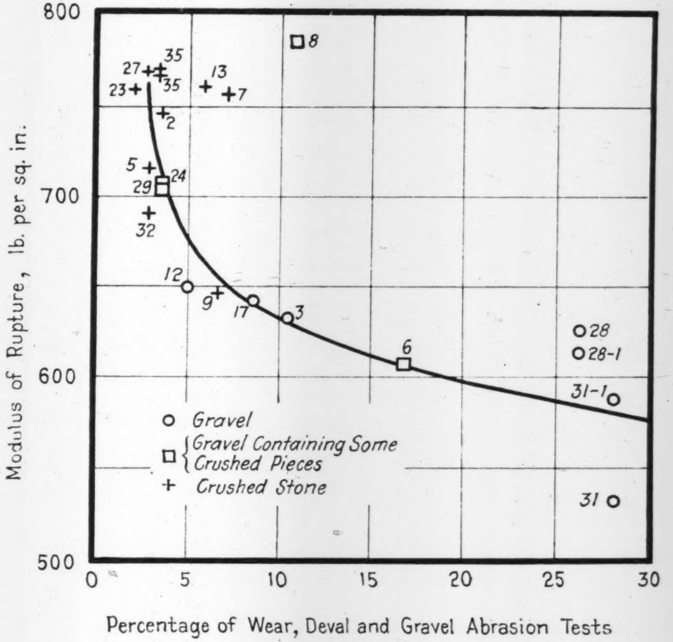

FIG. 1.—Effect of Percentage of Wear on Modulus of Rupture. 6.3 bags of cement per yard. Age, 28 days.

COARSE AGGREGATE IN CONCRETE

MR. GOLDBECK (*presented in written form*).—Mr. Lang has thrown some light on the question of the effect of the characteristics of the coarse aggregates on concrete strengths and he seems to show that although the physical characteristics as measured by the Deval abrasion test and the gravel abrasion test have little effect on the crushing strength of the concrete, they do have some influence on the cross-breaking strength. The writer has made special investigations along this line and the results are presented in the curves shown

TABLE I.—PROPERTIES OF CONCRETE, WITH DIFFERENT COARSE AGGREGATES GRADED AS RECEIVED.

Proportions 1:2:3½ dry, loose volume.
Age, 28 days—cured in moist room.
Slump, approximately 1½ in.

	Coarse Aggregate	Water-Cement Ratio	Flow	Weight of Unhardened Concrete, lb. per cu. ft.	Cement, bags per cu. yd.	Compressive Strength, lb. per sq. in.	Modulus of Rupture. lb. per sq. in.
No. 2......	Stone........	0.81	163	151.4	6.16	3760	703
No. 3......	Gravel.......	0.88	165	142.4	5.86	3630	558
No. 5......	Stone........	0.83	165	156.6	6.34	3670	674
No. 6......	Gravel.......	0.84	167	150.6	5.77	3450	535
No. 7......	Stone........	0.88	161	148.4	6.08	4300	720
No. 8......	Gravel.......	0.85	162	151.4	5.78	3960	678
No. 9......	Stone........	0.86	164	149.8	5.94	3600	640
No. 12.....	Gravel.......	0.85	167	147.6	5.89	3500	599
No. 13.....	Stone........	0.88	162	147.0	6.13	3800	666
No. 17.....	Gravel.......	0.81	164	148.4	5.82	3360	555
No. 23.....	Stone........	0.85	165	149.2	6.01	3810	717
No. 24.....	Gravel.......	0.81	165	153.6	5.89	3960	679
No. 27.....	Stone........	0.84	168	152.0	6.00	3680	686
No. 28.....	Gravel.......	0.88	168	147.6	5.59	3440	535
No. 29.....	Gravel.......	0.83	171	147.4	5.77	3840	622
No. 32.....	Stone........	0.86	161	147.8	5.88	3370	638
No. 31.....	Gravel.......	0.86	164	148.0	5.63	3160	517
No. 35-1...	Stone........	0.80	169	153.8	6.13	3620	701
No. 35-2...	Stone........	0.81	170	152.6	6.09	3770	712

TABLE II.—SHOWING PROPERTIES OF CONCRETE WITH DIFFERENT COARSE AGGREGATE GRADED AS RECEIVED.

Approximately same cement content and same consistency, age 28 days, cured in moist room.

	Coarse Aggregate	Proportions			Cement, bags per cu. yd.	Corrected Water-Cement Ratio	Slump, in.	Flow	Compressive Strength, lb. per sq. in.	Modulus of Rupture, lb. per sq. in.
		Dry Cement	Loose Sand	Volume of Coarse Aggregate						
No. 35.....	Stone.........	1	1.99	3.33	6.27	0.77	1.5	172	4490	768
No. 2......	Stone.........	1	1.99	3.33	6.30	0.78	1.75	176	4300	745
No. 3......	Gravel........	1	1.67	3.43	6.33	0.69	1.5	159	4750	633
No. 5......	Stone.........	1	2.19	3.29	6.28	0.83	1.25	169	4030	715
No. 6......	Gravel........	1	1.66	3.32	6.30	0.71	2.25	170	4310	607
No. 7......	Stone.........	1	1.92	3.32	6.32	0.77	1.5	164	4890	755
No. 8......	Gravel........	1	1.69	3.31	6.30	0.70	2.0	168	5020	786
No. 9......	Stone.........	1	1.83	3.30	6.28	0.79	1.25	159	4170	647
No. 12.....	Gravel........	1	1.79	3.31	6.30	0.77	1.5	163	4360	650
No. 13.....	Stone.........	1	1.92	3.36	6.34	0.78	2.0	164	4890	760
No. 17.....	Gravel........	1	1.73	3.32	6.30	0.68	2.0	164	4610	642
No. 23.....	Stone.........	1	1.87	3.32	6.31	0.78	1.75	162	4790	759
No. 24.....	Gravel........	1	1.79	3.34	6.28	0.69	2.0	165	4850	707
No. 27.....	Stone.........	1	1.92	3.28	6.30	0.80	1.5	166	4480	769
No. 28.....	Gravel........	1	1.51	3.36	6.32	0.71	1.5	160	4720	626
No. 29.....	Gravel........	1	1.71	3.33	6.31	0.70	2.0	166	4730	704
No. 32.....	Stone.........	1	1.79	3.41	6.27	0.77	1.75	163	4660	690
No. 31.....	Gravel........	1	1.57	3.41	6.27	0.70	1.5	163	4240	531
No. 28-1...	Gravel........	1	1.60	3.24	6.32	0.73	1.75	164	4480	613
No. 31-1...	Gravel........	1	1.64	3.29	6.29	0.72	1.75	160	4060	587
No. 35.....	Stone.........	1	1.98	3.29	6.30	0.76	1.75	164	4580	770

in the accompanying Figs. 1, 2 and 3. In these tests the concrete was proportioned in all cases so that it would have the same quantity of cement, 6.3 bags per cubic yard. The proportion of coarse aggregate used in the mix was determined somewhat after the method used by Talbot in which $\frac{b}{b_o}$ was made equal to 0.775

where b = solid volume of coarse aggregate in a cubic foot of concrete,
b_o = solid volume of coarse aggregate in a cubic foot of coarse aggregate.

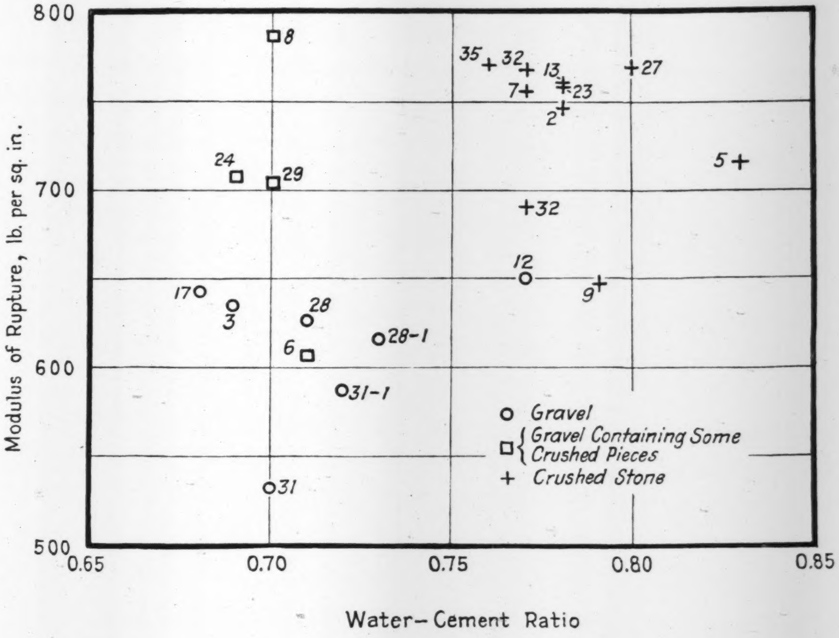

FIG. 2.—Relation of Modulus of Rupture to Water-Cement Ratio When Various Aggregates Are Used. 6.3 bags of cement per yard. Age, 28 days.

In all cases the consistency was made alike as nearly as possible as judged by the flow table and slump test. The 28-day strength results and the essential data of the concrete are shown in the accompanying Tables I and II.

Referring to Fig. 1, it will be noted that there is a general trend in modulus of rupture depending upon the quality of the coarse aggregate. Thus, those materials having exceptionally high percentages of wear have a low modulus. Those with exceptionally low percentages of wear have a high modulus. In Figs. 2 and 3 are shown the relation of modulus of rupture to water-cement ratio, and the crushing strength to water-cement ratio.

The noteworthy point about both of these curves is the fact that the same water-cement ratio does not necessarily give identical strengths when different aggregates are used and it is outstanding that a higher water-cement ratio may be used with angular coarse aggregate than with rounded coarse aggregates without any impairment in the strength. The two principal conclusions to be drawn from this series of tests are:

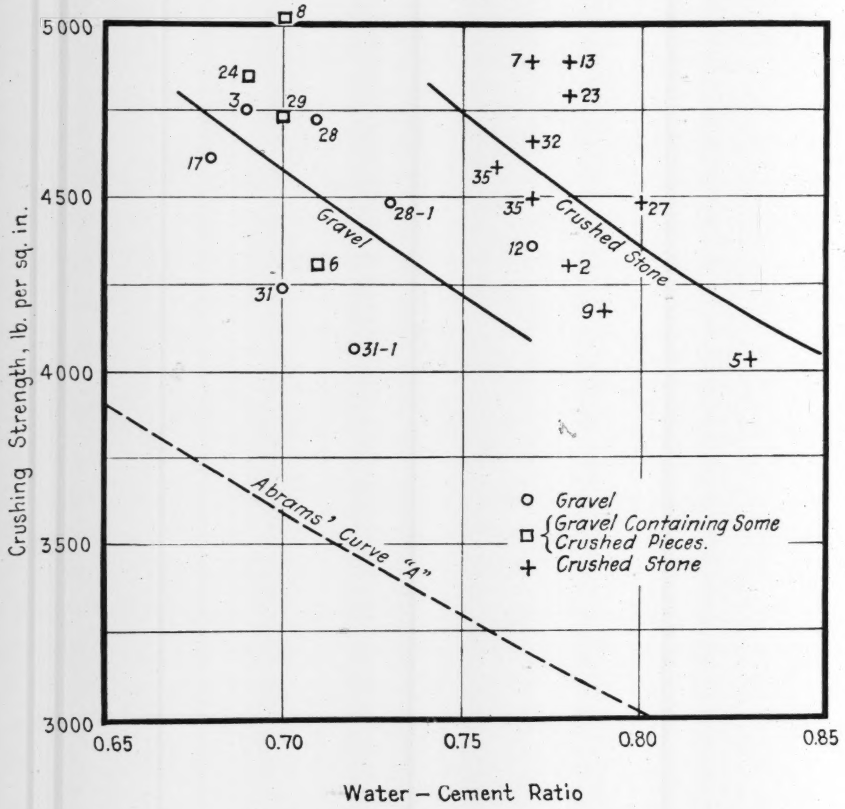

FIG. 3.—Relation of Water-Cement Ratio to Crushing Strength When Various Aggregate Are Used. 6.3 bags of cement per cubic yard. Age, 28 days.

1. The quality of the coarse aggregate as determined by the Deval abrasion test and the gravel abrasion test seems to have some influence on the modulus of rupture of concrete but does not have any effect on the compressive strength of concrete.

2. When different aggregates are used, different water-cement ratios are required to give the same strength of concrete. The angular aggregates, in general, require a higher water-cement ratio than the rounded aggregates. The important point to notice is, that if concrete is to be designed for a given strength, whether that strength be in cross-bending or in compression, it is not adequate nor economical to merely use an arbitrarily selected value for the water-cement ratio, but, rather, the water-cement ratio should be determined for the particular kinds of materials being employed.

MR. F. H. JACKSON[1] (*presented in written form*).—In connection with the general subject of Mr. Lang's paper, the U. S. Bureau of Public Roads has recently carried out quite an extensive series of tests for the purpose of determining the effect of type and gradation of coarse aggregate upon the flexural and tensile strength of concrete. These tests are reported in detail in the June, 1929, issue of *Public Roads*, the monthly magazine of the Bureau of Public Roads. It may be interesting, however, to call attention to certain of the outstanding conclusions which have been drawn as the result of this study.

The tests were conducted primarily for the purpose of determining the effect of type of coarse aggregate upon the flexural strength of concrete. Auxiliary data regarding resistance to direct tension as well as crushing strength and yield of concrete were also obtained. Seventeen typical coarse aggregates, including seven gravels, seven crushed stones and three blast-furnace slags were selected from various sources so as to give as wide a range in physical characteristics as possible. These were tested in concrete, using four typical coarse aggregate gradings as well as four paving mixes. The sand used was a typical high-grade concrete sand. The portland cement met all the requirements of the American Society for Testing Materials.

Each coarse aggregate was separated into four sizes at the laboratory and recombined into four definite gradings as shown below. These gradings will be referred to by number and it should be noted that they range from coarse to fine in numerical order of designation.

	PERCENTAGE PASSING EACH SIEVE (SQUARE OPENINGS)				
GRADING	⅛-IN.	¼-IN.	¾-IN.	1¼-IN.	2-IN.
No. 1	0	0	15	40	100
No. 2	0	0	30	55	100
No. 3	0	5	45	70	100
No. 4	0	5	55	100	100

All aggregates are identified by number rather than by source of supply.

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[1] Senior Engineer of Tests, U. S. Bureau of Public Roads, Washington, D. C.

In outlining this series of tests it was thought advisable to include more than one proportion, and for this reason four nominal mixes (based on dry-rodded volumes) were included, with the following proportions:

Mix No. 1.—1:1.6:3.
Mix No. 2.—1:1.6:4.
Mix No. 3.—1:2:4.
Mix No. 4.—1:2:4½.

Mixes Nos. 1 and 2 were designed to correspond approximately to 1:2:3 and 1:2:4 field mixes, respectively.

It will be noted that mix No. 4 (1:2:4½) is the only one which does not conform to present practice. This proportion was used in an endeavor to determine the manner in which the lower sanded mixes behave in flexure as compared to mixes containing smaller amounts of coarse aggregate, but with the same sand-cement ratio.

The accompanying Fig. 4 gives the average results of all strength tests for all aggregates except the three slags. They were omitted because they were not included in mixes Nos. 2 and 4 (1:1.6:4 and 1:2:4½) and naturally could not be included in these grand averages.

Referring first to the test for modulus of rupture, it will be observed that the aggregates arrange themselves in three general groups—the four aggregates to the right in the diagram comprising the high group, the six aggregates in the center the intermediate group, and the four aggregates to the left, the low group. Each of the points shown in this chart is the average of 64 individual tests. It will be observed that aggregates of a calcareous nature give higher flexural strengths than those of siliceous character. This is irrespective of whether the aggregate is a crushed stone or a gravel. Two of the aggregates composing the high group, it will be noted, are crushed limestone and two are limestone gravel. There is a maximum of 27 per cent difference in flexural strength obtained from the fourteen aggregates plotted in Fig. 4. A consideration of the water-cement ratio used for each aggregate would indicate that differences in water content cannot account for this difference. A study of all of the possible factors which might influence the results which are here shown would indicate that the mineralogical composition and surface texture of the aggregates are largely responsible for the strength characteristics developed.

In Fig. 5 are shown similar test data for each of the four mixes employed in the investigation. This chart shows the results obtained with the three blast-furnace slags to the extreme right of the diagram. It is interesting to note the relative small difference in flexural strength obtained with mix No. 3, 1:2:4, and mix No. 4, 1:2:4½.

In Fig. 6 there is shown the relation between aggregate types and a factor obtained by dividing the flexural strength developed with each aggregate by the corresponding cement factor. The resulting values may be used to compare the various aggregates on a strength-yield basis. Examining the curve showing the relation between modulus of rupture and cement factor, we note that there are four aggregates which stand out above the others. They are

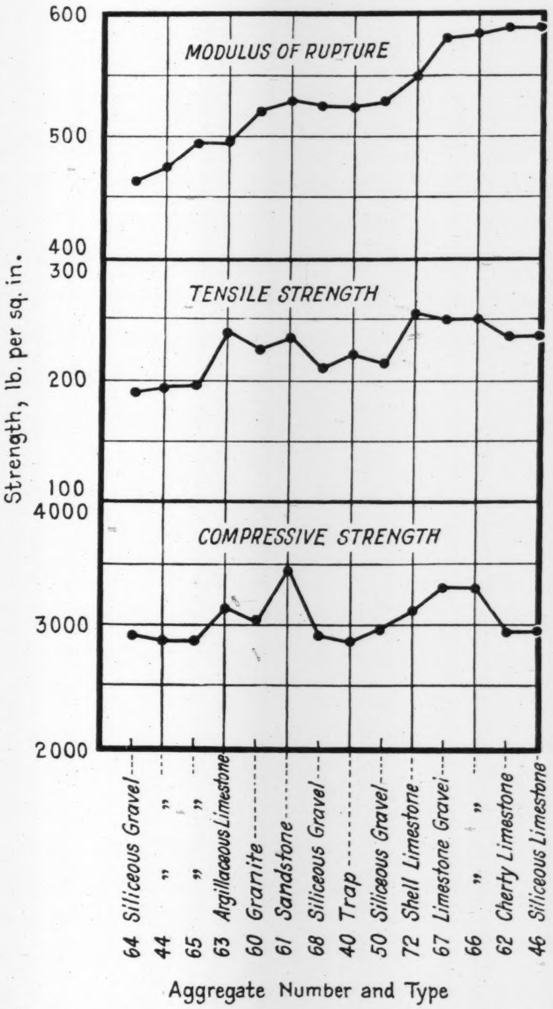

FIG. 4.—Curves Showing Strengths for Different Types of Aggregate.

Nos. 46, 62, 66, and 67 (two limestones and two limestone gravels), the same four that gave the highest flexural strength. There are four others, Nos. 50, 61, 68, and 72 which are grouped together while the remaining six, Nos. 40, 44, 60, 63, 64, and 65 show a rather low factor. With the exception of the trap and argillaceous limestone

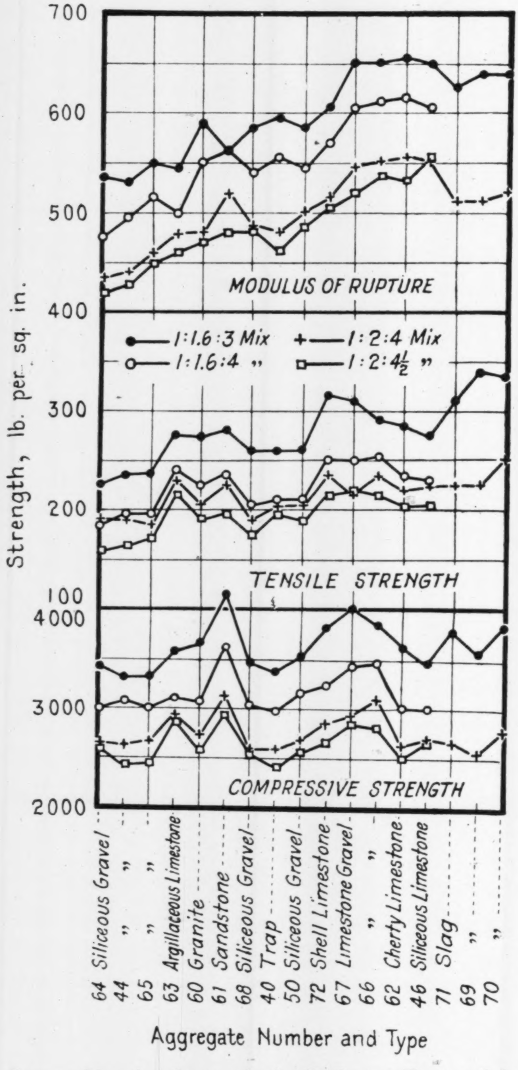

FIG. 5.—Curves Showing Strengths for Different Types of Aggregate in Each of the Four Mixes Employed.

aggregates Nos. 40 and 63, the aggregates line up in somewhat the same order as they do in Fig. 4.

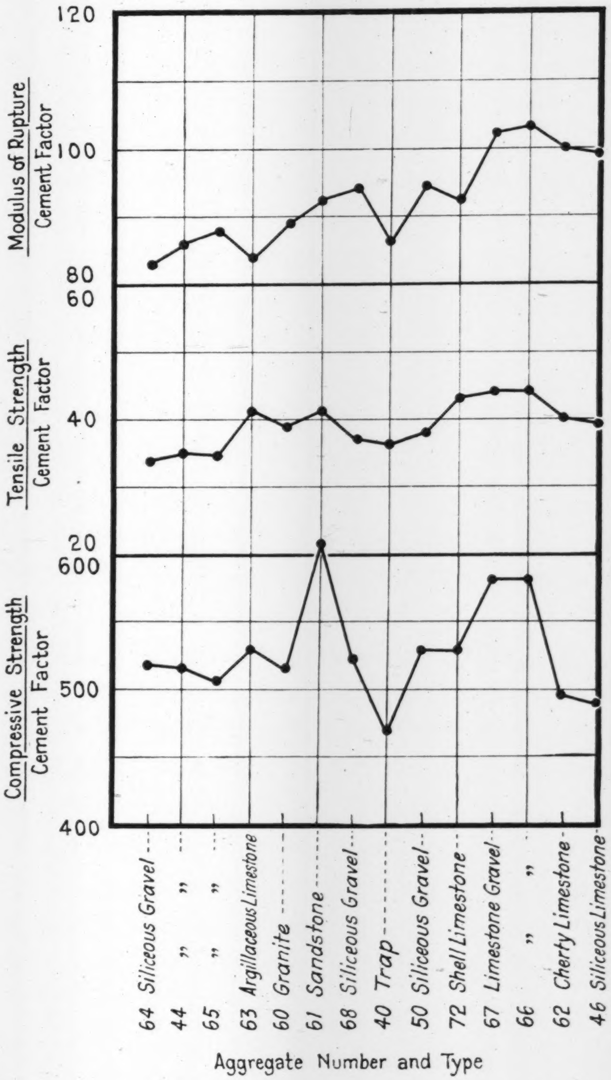

FIG. 6.—Relation Between Cement Factor and Flexural, Tensile and Compressive Strengths, Average of All Tests.

These facts demonstrate that the increased strength developed by certain aggregates is not due entirely to increased cement content in terms of unit of volume of concrete. If the variations in strength were due to this cause alone, we should expect the strength-yield factors derived in the above manner to produce a horizontal line, instead of an ascending curve.

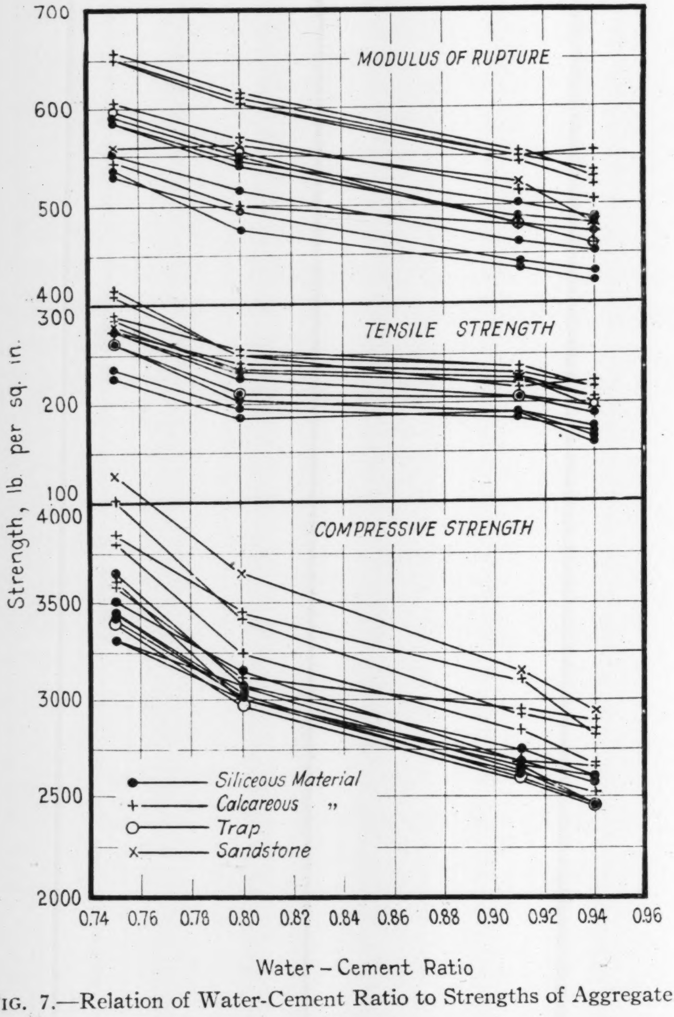

FIG. 7.—Relation of Water-Cement Ratio to Strengths of Aggregate.

In Fig. 7 there have been plotted the relation between the water-cement ratio used in these tests and the flexural, tensile and compressive strength developed for each aggregate separately.

It will be observed that for all three types of test the strength decreases proportionately with increases in the water-cement ratio, resulting in a series of substantially parallel curves each of which represents the strength - water-cement ratio relation for a given aggregate. For purposes of studying the effect of mineral composition the aggregates have been grouped into four divisions, the six calcareous materials, the single sandstone, the trap rock, and the various materials which are essentially siliceous, such as the quartz gravels and the granite. The grouping of materials by types as previously discussed is at once apparent. Of interest, also, is the fact that, in so far as tensile strength is concerned, the curves are practically horizontal for water-cement ratios between 0.8 and 0.94. Aside from this the relation between strength and water-cement ratio follows the well-known law fairly closely for each type of test.

The fact that these water-cement-ratio - strength curves are practically parallel gives us a method of designing concrete paving mixtures by trial as described in the paper, "The Design of Concrete Paving Mixtures by the Water-Cement Ratio Method," which appeared in the August, 1928, issue of *Public Roads.* This method consists essentially in determining for any given water-cement ratio, say 0.8, and for each combination of aggregates under consideration, the flexural strengths at 28 days under standard laboratory conditions. The resulting values are then plotted and curves drawn through each parallel to the basic water-cement-ratio - strength curve.

By noting the water-cement ratio at which each of these curves cuts the design strength line, the proportions which must be used in the case of each aggregate to secure the design strength with the desired workability may be determined by trial from which in turn may be determined the quantities of materials in each case for a unit volume of concrete. A study of the relative costs of materials will then fix, in general, the most economical combination to use.

Conclusions.—Based on the variables included in this investigation and the resulting test data, the following conclusions are drawn:

1. That the tensile, flexural and compressive strength of concrete are affected appreciably by the character of the coarse aggregate used.

2. That the tensile and flexural strength are affected to a greater extent than the compressive strength.

3. That for a given aggregate there is a fairly well-defined relation between the strength of the concrete and the water-cement ratio.

4. That variations in the character of the coarse aggregates, other things being equal, may result in a difference in flexural strength equal to that produced by an appreciable change in the water-cement ratio with any given aggregate. (In this study, for instance, aggre-

gate No. 46, with a water-cement ratio of 0.94, produced concrete of somewhat higher flexural strength than aggregate No. 44, with a water-cement ratio of 0.74—a difference of 0.2.)

5. That there is a fairly definite relation between certain mineralogical characteristics of the coarse aggregate and the strength of concrete, calcareous aggregates in general giving consistently higher flexural and tensile strength than siliceous aggregates.

6. That, in general, aggregates having rounded fragments produce concrete of lower flexural and tensile strength than aggregates which are composed wholly or in part of crushed fragments.

7. That, within the limits of this study, variations in grading of coarse aggregates have no consistent effect upon the strength of concrete. (It is not to be inferred from this statement, however, that control of grading is not important. Variations in grading occurring during construction not only affect yield when measurements are made by volume but also affect the workability and therefore the uniformity of the concrete.)

8. That, within the range in quality covered by this study, there is no relation between the quality of the coarse aggregate, as measured by the abrasion test, and the strength of the concrete.

MR. F. C. LANG.[1]—I understand from Mr. Jackson's discussion that calcareous aggregates in general give higher flexural and tensile strength than siliceous aggregates. I should like to ask if that implies that the same strengths were secured with the smooth particles as with the angular particles of the same material. If that was the case, that is, with the same quality of limestone according to absorption and other factors, one a gravel and the other an angular crushed stone, would the flexural strength be the same?

MR. JACKSON.—The statement referred to by Mr. Lang must be considered in connection with the sixth conclusion which is as follows: "That, in general, aggregates having rounded fragments produce concrete of lower flexural and tensile strengths than aggregates which are composed wholly or in part of crushed fragments." Our tests have indicated that angularity does have an effect upon flexural strength. The mineral composition, angularity, texture and other factors, I believe, all have an effect upon the flexural strength of concrete. Referring again to Fig. 4, two of the four higher values for flexural strength represent concrete containing gravel as coarse aggregate and the other two represent concrete containing crushed stone as coarse aggregate; in other words, there was no indication in this series of tests to justify a statement that, simply because a material is called gravel, concrete in which it is used should have a higher or a lower flexural strength than crushed-stone concrete or *vice versa*. The only point we desire to bring out is the fact that each individual aggregate must be studied and its characteristics noted in connection with the design of concrete mixture for flexural strength.

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[1] Engineer of Tests and Inspection, Minnesota Highway Department, St. Paul, Minn.

MR. MATTIMORE.—I want to say a word relative to the absorption of the coarse aggregates. If intelligently used, I believe the absorption has considerable value, but that means that we must classify our rock; in other words, if we take a so-called trap rock or diabase, the mineral construction is such that we should get a low absorption, and when we get a high absorption in such rocks it indicates that there is some deterioration, something different than is usual in a trap rock. Now, if we compare that with certain grades of sandstone, we have got something entirely different. The ordinary sandstone has high absorptive value, and it would not be correct to say that absorption always indicates structural weakness, unless we associate it with geological classification of the rock. It is the same way with limestone. In our studies of some of the sound aggregates in our state, we found that absorption values, provided we kept them in their own class or rock, was a very good indication of the state of preservation, and sometimes led to indications whether it was unsound material or not.

MR. MEYER HIRSCHTHAL.[1]—It has struck me that due to the general acceptance of the water-cement ratio, engineers have been less particular in the selection of their materials and bank-run gravel has very generally been used. It is really a reversion to bad practice, and while I understand perfectly that the appearance of organic matter is more generally common in loamy material, sand, there is still a great possibility of the appearance of tannic acid in gravel that is bank run; in fact, in one case I took the trouble to take a sample of bank-run gravel to use on a job and ran a colorimetric test on it and found that it was very highly colored. The point I want to bring out here is that there is a possibility of the necessity for putting a colorimetric test in the specifications for coarse aggregates where the coarse aggregate is bank-run gravel.

MR. BAILEY TREMPER[2] (*presented in written form*).—Mr. Lang comments on the lack of data regarding the influence of surface texture of aggregates on the strength of concrete. It has been the experience of the Washington State Department of Highways that surface texture is a very important characteristic, particularly if the concrete is to be used in a pavement.

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[1] Concrete Engineer, Delaware, Lackawanna and Western Railroad, Hoboken, N. J.

[2] Engineer of Tests, State of Washington, Division of Highways, Olympia, Wash.

There are in this state large bodies of gravel of glacial origin of very uniform character except in one particular, the condition of the surface. Infiltration of ground water has at places left a stain which in my opinion should be classed as a characteristic of the gravel itself and not as a coating because no amount of washing will remove it. Numerous tests have invariably shown that this stain causes a reduction in concrete strength both in compression and flexure. The percentage reduction is usually about twice as great in flexure as in compression. As the amount of mixing water required for a given consistency is not affected by the presence of this stain, the reduction in strength is believed to be due to a lack of bond with the cement paste.

Another instance of the effect of surface texture was observed during the examination of a river bar gravel. About 50 per cent of the material from this bar consisted of hard, water-worn pebbles of extreme smoothness. The pebbles had acquired sufficient polish to appear practically smooth to the eye. Under a 12× lens a certain amount of roughness was visible. A quantity of this gravel was hand-picked to secure a sufficient quantity of the smooth pebbles for compression and flexure tests. Comparative tests were made at the same time with a glacial gravel having well-rounded pebbles of excellent quality and of average surface texture. It is believed that the quality, aside from surface characteristics, was about equal in the two gravels.

In preparing concrete test specimens both gravels were combined to the same grading, and the same sand was used as a fine aggregate. The same mix, both by volume and weight, was used and both mixes required the same amount of water for equal consistency. Five specimens each were molded on different days.

The mix used was nominally 1:2:3 and contained 1.67 barrels of cement per cubic yard of concrete. At 28 days the specimens containing the smooth river bar pebbles had compressive and flexural strengths of 85 and 74 per cent, respectively, of that obtained with the pebbles of normal surface characteristics.

It is believed that these values represent the maximum effect due to smoothness of any aggregate likely to be encountered. The data should serve as a guide in evaluating aggregates of more than average surface smoothness.

MR. LANG (*author's closure by letter*).—The discussion gives data which were not available at the time the paper was written. On the whole they are supplementary.

I do not entirely agree with Mr. Goldbeck when he says that there is a general trend in modulus of rupture depending on the quality of the coarse aggregate. If he means to express quality by percentage of wear, I do not think there is a general trend. My opinion based on data now available is that the abrasion test on aggregates is only of value inasmuch as it may show aggregates of low structural strength. Aggregates with very high abrasion loss would probably pull apart so easily that the flexural strength of concrete would be appreciably affected. There is, however, a considerable range in percentage of wear for aggregates of suitable structural strength. For such aggregates the difference in modulus of rupture of the concrete may be due to other items, such as the difference shown in Mr. Goldbeck's curves between chert and limestone gravels.

My general conclusion is that the present tests, with proper test limits, may be depended on to furnish aggregates which are of suitable quality for concrete but that there is a considerable variation in the concrete making properties of the various aggregates which come within the ordinary specification limits. In order to take advantage of these on any particular job, it would be necessary to establish in the laboratory water-cement ratio curves for the aggregates and cement which will be used. The concrete test specimens should be tested according to how the concrete is to be used, such as flexure tests for concrete in concrete pavement. When different aggregates and cement are in competition, the most economical could be ascertained. The water-cement ratio law holds only for any particular type of aggregate. The durability of aggregates is very important, but was covered elsewhere.

SOUNDNESS OF AGGREGATE

MR. MATTIMORE.—The unsoundness of coarse aggregate is a subject in which I have been interested for the past ten years. Fortunately the engineering field is now taking some recognition of this subject, but my opinion is that we are complicating it by tying it up with so much technical details, such as the geological and mineralogical terms employed in the paper by McMillan and Ward. If rock is unsound it is susceptible to change or disintegration in nature. This can be observed in exposed ledges. In this regard I should like to give one precautionary note: An investigator in a new section should familiarize himself with the action of the native stone under weather conditions by examining exposed ledges. I have great faith in laboratory tests, but this is one place where we may not get full information relative to the ultimate durability of coarse aggregates. In our practice we do not accept material from any source in an unfamiliar location until complete examination has been made of the exposed ledges. It may be advisable, at least interesting, to take this subject up from a mineralogical standpoint, but if this practice is found necessary it places it beyond the field of the average engineer. We know from past experience that unsound rock can be identified by examination of exposed ledges, and from that standpoint why not perform this operation in the simplest manner.

MR. S. H. INGBERG.[1]—I should like to ask Mr. Mattimore if, in observing the exposed ledges, he would make any allowance for the age of the ledge? Does he mean a natural exposed ledge, or one exposed by quarrying operations?

MR. MATTIMORE.—In regards to Mr. Ingberg's question the exposed ledges referred to are the natural ledges or quarry sections which had been exposed for a number of years. Naturally the extent of the exposure of the ledge would affect the condition of the stone, but we have found that stones of an unsound character which are likely to have a detrimental effect in construction show signs of disintegration when exposed for four or five years to the elements.

MR. LANG.—I wish to emphasize Mr. Mattimore's statement about examining ledges. It has been our practice to do that for several years. I do not think that one can lay down any fixed rules. The man who is examining must be a man of experience, he must examine every ledge, and I do not see that it makes any difference whether it is a natural face or a quarried face, it is to see what nature has done to that material over a period of years. I do not think that any material, especially of sedimentary rock, should be used unless the engineer has examined the face or knows that it has been used successfully in concrete over a period of years. I think it is the volume change which has taken place in the rock during the disintegration that is most important. Such aggregate when placed in concrete exposed to the elements increases in volume and during this action cracks and disintegrates the concrete.

MR. HAROLD ALLEN.[2]—I want to agree with Mr. Lang and Mr. Mattimore in the matter of examining the ledges in stone quarries for evidence of unsoundness. I wish to state, however, that in Kansas we have had considerable success with the actual freezing-and-thawing test for soundness and we find that the results of that test check out very well with the examination of the ledges. We regard the sodium sulfate test as unreliable and we have gone over almost entirely to the alternate freezing and thawing method.

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[1] Chief, Fire Resistance Section, U. S. Bureau of Standards, Washington, D. C.

[2] Materials Engineer, Kansas State Highway Commission, Topeka, Kans.

MR. R. B. GAGE.[1]—Listening to what has been said about chert and the effect it may have when used as an aggregate in concrete, and knowing the close relation between chert and flint, it certainly is very confusing to try to straighten out the relative values of these materials from a road builder's standpoint. The mineralogist classifies both of these materials under the same heading and practically as the same thing.

It appears there has been some trouble with certain limestones which contained chert, yet the best gravels we have in New Jersey are full of pebbles of flint, which is one of the hardest, densest and most durable rocks to be found in the gravel. To date we have never noticed any deleterious effects of this particular ingredient in the gravel and we certainly would hardly be justified in rejecting gravel that contained flint pebbles. It may be that the material designated as chert may not be a real chert but a kindred material that will weather and go to pieces when exposed to the elements.

It is a well-known fact that certain varieties of opal will slowly dehydrate when exposed to the air, which ruins them for gem purposes. All of these materials are hydrated minerals which may or may not lose their water of hydration when exposed to the atmosphere, depending upon the relative character of the material itself. Certain shale rocks which outcrop in New Jersey and extend northeast and southwest from Massachusetts or from New York into Alabama are apparently stable enough when they are not exposed to atmospheric conditions, but quickly crumble and go to pieces when thus exposed. The freshly mined material will show a nice fractured surface, and, as Mr. Mattimore suggests, it is not always safe to base our conclusions regarding the durability of a rock upon such a fractured surface. Neither should we assume that the age of a rock from the geological standpoint has much bearing upon its durability, for the trap rocks in New Jersey are one of the latest formations, or rather youngest of the rocks, yet they apparently are one of the most durable. Again, there does not appear to be any good reason to require the use of rock from a weathering point of view that will not disintegrate in from 50 to 100 years when we are quite positive that the life of a pavement in which it may be used will very seldom be more than 20 years.

MR. H. F. KRIEGE.[2]—Chert has been mentioned several times, and there has been some feeling that chert disintegrates, but the kind that disintegrates does not occur in gravel. It is certainly true that in the formation of gravel from an exposed limestone or other type of stone ledge, there is every opportunity to permit the disintegration to take place before the rounding of the particles into gravel has been completed. However, some gravels from Indiana, whose composition is probably more than 50 per cent calcareous, contain chert which will disintegrate under the soundness test. They will disintegrate either alone or in concrete, so that the occurrence of chert which *will* disintegrate—I am making that distinction purposely—in gravel, is just as real as in limestone, though the chances for its appearance are much less, because the natural forces of weathering have made for a survival of the fittest.

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[1] Chemical Engineer, New Jersey State Department of Conservation and Development, Trenton, N. J.

[2] In Charge of Tests, France Stone Co. Labs., Toledo, Ohio.

MR. WALKER (*by letter*).—The paper by Messrs. McMillan and Ward gives a very excellent discussion of the influence of the coarse aggregate on the durability of concrete. It includes, however, a somewhat categorical statement concerning the effect of flat particles in concrete which, it seems, should be qualified to some degree. Tests recently carried out in the Research Laboratory of the National Sand and Gravel Association furnish information on the concrete-making properties of one gravel containing flat particles up to 15 per cent. The results of those tests were presented before the present annual meeting of this Society.[1] For the conditions of the tests, as much as 15 per cent of the flat particles had no appreciable effect on the strength or workability of concrete. Determined efforts to float the flat particles to the top of slabs 4 in. thick failed to result in any higher proportion of flats on the surface than were distributed throughout the mass.

The authors also point to pit holes in the surface of concrete as contributing to ready disintegration and as being particularly deleterious on wearing surfaces. The writer has made careful inspections of a great many road slabs containing pits caused by soft particles of aggregate which showed no evidences of disintegration or excessive wear.

MR. GOLDBECK (*presented in written form*).—The question of soundness is a highly important question at the present time. Many materials are open to doubt, and the fact is that there is no adequate soundness test which will permit an engineer to say definitely that a material is unfit for use. The sodium sulfate soundness test, of course, gives very strong indications, but, on the other hand, it may give very erroneous indications; generally, on the side of safety but apparently not necessarily so.

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[1] Stanton Walker and C. E. Proudley, 'Effect of Flat Particles on Concrete-Making Properties of Gravel," Appendix to Report of Committee C-9 on Concrete and Concrete Aggregates, *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part I, p. 322 (1929).

Some interesting experiments were made by the writer to determine the effects of certain variations in the method of making the sodium sulfate test and it was found that depending upon the temperature at the time the solution was prepared, upon the temperature during the time of immersion, upon the rate of evaporation of the solution and upon other variables, the amount of crystallization which took place in the solution varied tremendously. There was no definite relation, however, between the amount of crystallization and the destructive effect on the sample. Any of the variations made in the test are possible under the present very inadequate method of describing this method so that the test results obtained in various laboratories are very frequently different. In the accompanying Tables III and

TABLE III.—VARIATIONS IN PROCEDURE FOR MAKING SODIUM SULFATE TEST.

Test	Temperature of Solution at Immersion, deg. Fahr.	Presence of Crystals in Solution Before Immersion of Sample	Pans Covered or Uncovered	Temperature Condition of Rock when Immersed
No. 1......	90° solution saturated at 90°..	No excess crystals.....	Uncovered....................	Hot
No. 2......	70° solution saturated at 70°..	No excess crystals.....	Uncovered, temperature 60° F..	Hot
No. 3......	70° solution saturated at 70°..	No excess crystals.....	Uncovered....................	Cool
No. 4......	70° solution saturated at 70°..	No excess crystals.....	Uncovered, temperature 80° F.[a]	Hot
No. 5......	70° solution saturated at 70°..	Excess crystals........	Covered......................	Hot
No. 6......	70° solution saturated at 70°..	Excess crystals........	Uncovered	Hot
No. 7......	70° solution saturated at 70°..	No excess crystals.....	Covered......................	Hot
No. 8......	70° solution saturated at 70°..	No excess crystals.....	Uncovered....................	Hot

[a] Pan placed in moist atmosphere.

TABLE IV.—RESULTS OF SODIUM SULFATE TEST CONDUCTED IN EIGHT DIFFERENT WAYS.

Alternation Number	Test No. 1		Test No. 2		Test No. 3		Test No. 4		Test No. 5		Test No. 6		Test No. 7		Test No. 8	
	O.K.[a]	Failure	O. K.	Failure	O. K.	Failure	O. K.	Failure	O. K.	Failure	O. K.	Failure	O. K.	Failure	O. K.	Failure
1............	10	0	10	0	10	0	10	0	10	0	10	0	10	0	10	0
2............	10	0	10	0	10	0	8	0	10	0	10	0	10	0	10	0
3............	10	0	8	0	7	0	8	0	9	0	9	0	9	0	10	0
4............	10	0	7	0	6	0	8	0	9	0	5	0	9	0	10	0
5............	8	0	3	0	3	0	4	0	4	0	2	0	7	0	5	0
6............	8	0	0	0	0	0	2	0	0	0	0	0	1	0	1	0
7............	8	0	0	0	0	0	2	0	0	0	0	0	1	0	1	0
8............	6	0	0	0	0	4	2	0	0	0	0	0	0	0	0	0
9............	4	0	0	0	0	5	1	0	0	0	0	0	0	0	0	0
10............	4	0	0	0	0	5	1	0	0	0	0	0	0	0	0	0
11............	3	0	0	1	0	5	1	0	0	0	0	0	0	0	0	0
12............	3	0	0	1	0	5	1	1	0	0	0	1	0	0	0	0
13............	3	1	0	3	0	6	1	2	0	3	0	4	0	0	0	0
14............	3	2	0	4	0	8	1	4	0	4	0	4	0	0	0	0
15............	3	2	0	4	0	8	0	4	0	5	0	5	0	0	0	0
16............	3	3	0	5	0	8	0	4	0	5	0	5	0	0	0	0
17............	2	3	0	5	0	8	0	4	0	6	0	5	0	0	0	2
18............	1	3	0	5	0	8	0	4	0	6	0	5	0	0	0	2
19	1	7	0	10	0	8	0	7	0	7	0	7	0	5	0	7
20............	1	7	0	10	0	8	0	8	0	8	0	7	0	6	0	8

[a] Unaffected by test.

NOTE.—Ten pieces used in each test. Pieces showing incipient failure are not noted in this table.

IV are given the results of these various test methods. Note that the sample failed in some of these methods and was entirely sound when tested in other ways. Incidentally, the particular specimen used, which is a soft marble, is known to be absolutely sound and has not failed after 75 alternations in a severe form of freezing test, when the stone was partially immersed in water. In this case the sodium sulfate test gives an entirely wrong indication of the value of the stone and this has been found to be so with other materials in which comparisons have been made.

A peculiarity has been noted in a number of instances where the sodium sulfate test has been made on samples of blast-furnace slag. Apparently, the same kind of crystals of sodium sulfate do not form in the solution in which blast-furnace slag is immersed as occurs with other materials. It has been noted that the crystals instead of adhering to the slag in a firm, hard, definite form, more nearly resemble slushy snow which lies in the bottom of the container. This is a subject which needs investigating to determine if there is not some chemical action going on which impairs the sodium sulfate test as a measure of the soundness of materials of this nature.

Incidentally, it is the writer's opinion that we are attempting to use one form of test as a single means for determining various kinds of unsoundness which the tests may not detect. The sodium sulfate test is intended to simulate ice action. Is it not true that there is unsoundness of materials due to factors other than ice action? The question of the different kinds of unsoundness which may exist in various aggregates should be given thorough consideration, and it may result in the formulation of several standard soundness tests, depending upon the kind of unsoundness to be expected.

MR. FRED HUBBARD.[1]—Referring to Mr. Goldbeck's comments on the sodium sulfate test as applied to blast-furnace slag, my observation of the results of the sodium sulfate test for soundness of slag is that there is no apparent sloughing of the slag in this test. The solution is usually clear and entirely free from slime of any kind removed from the slag or found on the surface of the slag. In a few instances the solution has been observed to have a very slight milky appearance which may be the condition observed by Mr. Goldbeck.

MR. INGBERG.—On the subject of the sodium sulfate test, I do not know but what there is possibility of an inhibiting action by the chemicals in the sodium sulfate test such that we get results not comparable at all with what results from weather exposure. Such effects have been observed in the case of burned-clay products the action concerned being, I believe, hydration of free lime. Its effects were not apparent in the sodium-sulfate test, whereas they appeared in the ordinary freezing-and-thawing test. Would there be a possibility of such an effect with the class of materials we are dealing with here?

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[1] Consulting Engineer, The Standard Slag Co., Youngstown, Ohio.

MR. CLARK.—In reference to the sodium sulfate test, perhaps attention has not been called to the fact that the maximum solubility of sodium sulfate, as we understand it, is at about 32° C. At temperatures above that, the solubility ceases; that is, the water of crystallization is different, the process is different from that which takes place below 32° C. Someone who is better versed in physical chemistry than I, can probably explain that crystallization and how different types of crystals may form different disintegration products. The disintegration of the stone surface may be profoundly affected by the type of crystal formed at different temperatures. We have practically abandoned the sodium sulfate test as a measure of the weathering quality of stone in Virginia. Our results were so variable, and in many cases so at variance with the known facts, that we were unable to correlate the sodium sulfate test with the durability of stone.

I have in mind one case where I investigated a gneissoid granite, that is a granite with gneissoid structure, and although the difference in the toughness at right angles to the plane of cleavage and along the plane was very slight, the bands were not very thick (probably 0.5 mm.) and there was a slight difference in the coloration from white to grayish white. This material seemed very sound, and when we put five pieces through the sodium sulfate test, only one showed any sign of disintegration at all, and that was very slight and on a rather thin section. The outcrop face was shot down to examine the uniformity and density of the deposit. This was done in October.

The following June, when I went to investigate the situation, the surrounding field was covered with pieces of stone, from 2 to 30 or 40 lb. in weight, that had been blown away from the face. Those pieces had been perfectly sound, apparently, when the shots had been put off, but in the following June practically every piece had started to slake; pieces about the size of one's hand were slaked as though they were burned lime exposed to the weather. The largest pieces were from 1 to 2 in. in size.

This stone was located near the North Carolina border, where the temperature in winter is not severe; I doubt if there had been any ice formed in that area that year. This was apparently a case of weathering due to wetting and drying, yet that stone had satisfactorily passed the sodium sulfate test.

We have had so many similar cases in different types of stone, because we have, in Virginia, every conceivable kind of stone, I believe, metamorphic, igneous and sedimentary, and they do not run true to form in so many cases with the standard tests to which we put highway materials, that the sodium sulfate test has been practically abandoned as a measure of the weathering qualities of stone.

MR. C. E. PROUDLEY.[1]—In making a sodium sulfate test, it might be well to use a little larger sample than is now required.[2] Ten pieces weighing approximately 1000 g. is hardly suitable for a stone which is not of uniform character throughout. Sometimes this test is used for gravel, and if it is non-homogeneous, it is difficult to get a representative sample in 1000 g. If one has only ten pieces in the sample, and one piece goes bad, that is 10 per cent of the sample that is unsound. Of course 10 per cent may not be a very high percentage of unsound material; however, I would suggest that, particularly in cases where the material is composed of a number of different types of minerals, a sample of possibly as much as 5000 g. and in some cases more, be used.

MR. LANG.—It has been my version for some time that the sodium sulfate test on rock for soundness might be compared, in a way, with the color test on sand, it is a warning test; it shows bad action, and this stone should not be used unless there is further investigation. It should be treated as a warning test in investigating a new source of supply, along with other investigations rather than as a direct specification.

MR. P. J. FREEMAN.[3]—I want to call attention to Mr. Kriege's reference to the work of Edward Orten on the proposed tentative specifications for building brick, and suggest that those who are interested in the sodium sulfate test read about the work done in 1919. The report mentions the fact that unless the specimen is placed immediately into a preheated oven, an entirely different type of crystal develops than if placed into a cold oven. It is a worth-while report with a number of pages of data on the soundness test.

In commenting on Mr. Lang's remarks that the sodium sulfate test should be used as a warning, you will recall that Mr. Loughlin of the United States Geological Survey did some work along that line, part of which was reported by Committee C-9. I am also calling your attention to the fact that the Portland Cement Association has employed Mr. Ward as a petrographer, who is in a position to cooperate with the users of aggregates and furnish definite geological information in connection with soundness of aggregates. I am sure we will appreciate the importance of this step taken by the Portland Cement Association in making the cooperation of the petrographer available for the study of the durability of concrete.

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[1] Assistant Director, Engineering and Research Division, National Sand and Gravel Assn., Washington, D. C.

[2] Proposed Method of Test for Soundness of Coarse Aggregate, *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 361 (1928).

[3] Chief Engineer, Bureau of Tests and Specifications, Department of Public Works, Allegheny County, Pittsburgh, Pa.

MR. KRIEGE.—Just one word in defense of the sodium sulfate test. A committee has been at work for the American Society of Civil Engineers to study the methods of testing the soundness of filtering materials. The published report of the A.S.C.E. committee is not yet available. As I recall the committee's plan, material was chosen which would fail before 5, some which would not fail until between 5 and 15, and some which would stand for 20 cycles without failure. About six laboratories were to carry on the same tests in the same manner with those materials. Four of them found that the material which failed before ten failed within one immersion in each case; that is, if one laboratory found it to fail at the fourth, the others found it to fail at either the third or the fifth. Now that is, I think, a fairly satisfactory duplication of results by different laboratories using the same method. We have heard this morning that the abrasion test can be interpreted in different ways for the concrete-making properties of aggregates, likewise hardness, toughness and the other physical properties. That means that neither of those characteristics is a definite criterion for quality. I feel that if the soundness test points the way to determine materials of questionable durability, it should not be disregarded. In some cases we know it does not work.

With regard to the suggestions that were made as to how this test should be conducted, I would like to add one further thought to Mr. Goldbeck's fine presentation. We are thinking in terms of the soundness test as an accelerated test. I believe it would be of definite value if we could cut down the length of time to ten cycles. If we could make ten cycles as severe punishment as twenty, we would need to wait only a week and a half instead of three weeks, and that would be no small contribution in itself. To that end I would say that it would probably be easier to control a temperature range through which the bath should form than it would be to keep it at a constant temperature. As has been pointed out, now the effect of the temperature at which these specimens go into the solution is very pronounced; you cannot keep the sodium sulfate bath from rising in temperature when you immerse into it specimens of stone that have been heated to 100° C. If the specimens are always heated to the same extent, the chances are that the temperature rise of the solution would be approximately the same in each case. Therefore I am bringing out this thought, to include the idea of making the test more severe, in order that the conclusions may be arrived at a little sooner.

AGGREGATES FOR RAILROAD BALLAST

MR. WALKER.—As Mr. Kriege has pointed out, railway ballast provides one of the largest fields of use for mineral aggregates. While the desirable characteristics which a ballast should possess are fairly well known, information permitting the measurement of these characteristics or the preparation of specifications which will insure their presence has not been developed fully.

The Sub-Committee on Washed Gravel Ballast of the Committee on Ballast of the American Railway Engineering Association has formulated tentatively definite specifications for grading of washed gravel ballast and is carrying out work looking to the development of test methods for controlling other desirable properties.

The Research Laboratory of the National Sand and Gravel Association is cooperating in this work and an investigation has been outlined which has for its principal purpose the development of test methods which will be indicative of the hardness, strength, and durability of gravel ballast. Other tests under way and projected have for their object the determination of the relationship between grading of gravel and its void content, and the securing of information on what properties of gravel ballast affect its resistance to movement within itself.

A measure of the hardness and strength of the gravel should give some indication of its resistance to breaking up in the track due to train loads and impacts. Abrasion and crushing tests are being carried out in an effort to develop criteria for these properties. A test which will give information with reference to the durability of gravel, when correlated with the behavior of the gravel in service, should be of value in forming an opinion as to its ability to resist the action of weathering.

In the case of the tests for voids and resistance to movement, it seems reasonable to suppose that the data will be of assistance in evaluating the effect of grading on stability, drainage and, perhaps, other properties. Probably the lowest void content consistent with proper drainage will furnish a ballast of the greatest stability feasible to use in practice. It is hoped that the tests on friction or stability will help to develop this point.

NEEDED RESEARCH ON MINERAL AGGREGATES

MR. A. N. TALBOT.[1]—I think we all are appreciative of the matters that Mr. Jackson has brought to our attention in this paper on needed research on mineral aggregates, and the occasion should not go by without voicing a word of appreciation of the thoughts that he has given in this paper. He calls attention to the fact that fundamental knowledge regarding properties and uses of mineral aggregates has not kept pace with the actual use of such materials in construction —I am quoting from the synopsis—and further that "the need for a correlating agency to review and analyze existing data as well as to coordinate future activities along this line is stressed." And his conclusion ends with a further reference to the same idea. It seems to me that the occasion should not go by without asking whether there is something which may be done in this direction which is not now included in the work of the committee? I wonder whether Mr. Jackson does not have some method or some thought as to what steps should be taken at this time to put such matters into effect. I desire to ask him that question.

MR. JACKSON.—I certainly have a very definite thought as to the need for such action. I must confess, however, that I cannot give at the present time any very definite recommendations as to how it should be accomplished. So far as highway material is concerned, I feel that we already have in the Highway Research Board an agency which can do very excellent work along that line. The Highway Research Board is created for just such a purpose. I recall that one or two years ago an effort was made to arouse interest in the subject through Committee E-9 on Correlation of Research. However, there was some difficulty in securing the cooperation of other organizations representing users of aggregates. Partly as a result of those discussions of the various problems involved, the present symposium has developed. I cannot but feel, as I have emphasized in this paper and as I think you feel, that there is primary need for some action along this line.

MR. H. J. GILKEY[2] (*by letter*).—In spite of an enormous volume of published data purporting to show one thing or another about the influence of aggregates on the properties of concrete, the present status is far from satisfactory.

The more indefinite the standards of excellence, the greater are the amounts of skill, judgment and diplomacy that must be exercised by the engineer or the inspector. Tolerances must also be more liberal. In his introduction to this symposium, Mr. Crum has brought out the fact that the man who can equitably inspect aggregates, in the face of the existing vagueness of present knowledge, is indeed qualified for better things.

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[1] Professor Emeritus, University of Illinois, Urbana, Ill.

[2] Associate Professor of Civil Engineering, University of Colorado, Boulder, Colo.

Without detracting from the importance of the points considered by Mr. Jackson, it seems to the writer that the time is now ripe for aggregate researches of a new variety. Thus far tests of aggregate in its relation to the properties of the concrete have always had present many simultaneous variables. While some of these may not be important in their bearing upon the resultant properties, the possible effect of others has been lost in the maze of conflicting influences.

The result is a lack of definite knowledge regarding any one of many possible influencing factors. Take, for example, the subject of flat or elongated particles. Most coarse aggregates contain some of these. How many are too many? how flat or how elongated need they be to be objectionable? how much extra cement would offset their weakening effect? etc. What sound quantitative basis has an inspector, engineer, or specialist for the exercise of judgment on this or on a score of other important points?

Suppose it be granted that with sound, durable, inert aggregate materials, the aggregate as generally used will be a secondary factor in its effect upon strength and other properties. Nevertheless there is some reason to believe that the grading, shape, size or surface texture of some aggregates may influence the compressive strength as much as 15 or 20 per cent. The effect on flexural strength may be even greater. With an accumulation of effects, instead of a compensation, the influence of an aggregate might be much more pronounced.

The use and development of concrete has proceeded so rapidly that the only practicable course has been to focus upon the high spots. If sound, durable, inert aggregates of random gradings, surface textures, shapes, and strengths gave concretes within 20 or 30 per cent of one another as judged by strength, cost or other standard of comparison, it was proper to ignore secondary factors for the time being.

Control of water and cement and proper placement and curing are the major factors that influence appearance, strength, serviceability and durability and it is these that have 50 and 100 per cent effects.

Is it not possible that conditions now call for supplementary researches along less practical but possibly more basic lines?

Concrete is made up of cement paste and aggregate. The *initial* or *primary* failure must occur in the mortar, in the aggregate or at the junction of the two.[1] Much work has been done and much of a basic nature has been learned about the paste. Not much is known about the bond between aggregate and paste as determined by shape, size, and surface texture. Nor is there much evidence on such individual factors as the effect of the strength, stiffness, grading and arrangement of the individual aggregate particles when embedded in different mortars.

To segregate the variables it would be necessary to definitely control the quality, shape, surface texture, and arrangement of the aggregate particles used, in addition to the quantities. To accomplish this, many freak or unusual aggregates would probably have to be resorted to in order to exaggerate and explore limitating cases. Metals of variable specific gravities, sizes, shapes, and surface textures and treatments could be employed in various proportions and arrangements. Precast cement or mortar aggregates of controlled shapes, strengths, sizes, etc., could also be used to cover other cases.[2]

Such basic trends, once determined, would exclude from consideration neutral factors or nonpertinent variables, and would give a solid groundwork for the development of tolerances or more definite specification clauses, in some cases, and for the exercise of sounder judgment in others.

In this discussion it is not the aim to detract from the excellent program that Mr. Jackson has outlined. These are suggested additions and not substitutions. Consideration of the points mentioned is a task for the laboratory technician or research worker. The practical concrete operator already has his hands full if he watches major factors and builds to the best of current knowledge.

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[1] An inspection of a failed specimen is likely to give little indication of the primary cause of failure. Following initial slippages or yielding there is a progressive breakdown that will wreck many elements that were amply strong until other parts gave way. An example of this fallacy is the widespread belief that the mortar is as strong as the aggregate if some of the aggregate fractures. Often the plane of fracture is determined by other elements and a well-anchored fragment will break across if it is easier for it to do so than to be pulled out or if the fracture would have to traverse a longer path in the mortar. Wedging and splitting also enter.

[2] Such an investigation is outlined in greater detail in a former paper by the writer, ***Proceedings***, Am. Concrete Inst., Vol. XXIII, pp. 381–386 (1927).

FUNDAMENTAL FACTORS IN THE TESTING OF MINERAL PRODUCTS WITH SPECIAL REFERENCE TO SLATE AND RELATED MATERIALS[1]

By Oliver Bowles[2]

Synopsis

Products of mineral origin are divided into two groups: (1) those made from crude raw materials that require smelting or other treatment, and (2) those that retain their natural structure and properties in the finished product. Examples of the first group are metals and alloys; of the second group, slates and building stones. In testing and establishing standards for materials in the first group, modifications in properties and composition can usually be made. In the second group no such modifications are possible, and adaptation for use must be accomplished by selection only; selection for use is therefore of paramount importance in this group.

Manufacturers have before them the prodigious task of testing and classifying these natural products and of diverting them to the uses for which they are best adapted. Problems of adaptation to use are complicated by the interdependence of materials used in conjunction with each other; that is, the standards established for any one product are dependent upon the requirements of use in association with other materials. This involves overlapping of committee work and demands systematic correlation of interlocking activities.

Products of mineral origin may be divided into two distinct groups depending upon whether the inherent physical properties are altered in the process of preparing them for industrial use. For one group nature has supplied raw materials in such a crude form that they must be smelted, refined, compounded or treated in various ways to form finished products differing entirely from the original minerals or ores. Examples of products in this group are manufactured chemical salts, metals, glass and pottery. These synthetic or refined products may within certain limits be changed in composition, texture, strength and other properties. The second group consists of materials that are used in the form in which they occur in nature, the only modifications being in the size, shape, and possibly the surface finish of the natural product. Sheet mica is a good example of the materials in this group, for man may split it to any desired thickness, and shape it into any desired forms, but he can not improve the quality nor change the physical or chemical properties of the mica as it originally occurred. Other examples are slates and building stones which are used as materials of construction with the same composition, texture, and physical properties as nature gave to them.

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[1] Published by permission of the Director of the Bureau of Mines of the U. S. Department of Commerce, Washington, D. C.

[2] Supervising Engineer, Building Materials Section, U. S. Bureau of Mines, Washington, D. C.

There are, of course, certain products which fall on the border line between these two groups. For example, asbestos must be fiberized before it is spun into yarn or woven into cloth, and a sandy clay must be mechanically freed from its impurities before it is used as a paper filler. For these materials some treatment process is required, but the natural properties are essentially unchanged.

The American Society for Testing Materials has for many years worked through its committees in testing and determining specifications for various materials in both of these great groups, mostly those in the first group, but the fundamental differences in the mode of attack have never been sufficiently emphasized.

The testing of synthetic or refined products belonging to the first group is interwoven with problems of compounding and treatment. If certain desired properties are not in evidence, the composition or treatment may be changed until the properties sought are attained. The manufacturer may change the proportions of the constituents, he may add new metals or he may alter the properties of the compounds formed; he may plunge the heated product into water or oil to temper it to a fixed degree of hardness. Thus man has the power within certain limits of changing the properties of these materials at will in order to adapt them to specific uses.

When we consider the second great group, of which slate and other structural stones are types, the problems are entirely different. Tests must be made and specifications must be established, but the manufacturer is helpless when confronted with the task of changing the properties of his materials. Nature has been the fabricator, the great geological mountain-building forces were the power plants, the folding rocks were the machines, and the internal heat of the earth, or the heat induced by intense pressure, constituted the furnaces in which these materials were refined, purified and crystallized. To-day there is an insistent demand for the latest and most approved methods of manufacture, but he who deals in these natural products must accept what Nature herself has manufactured by methods which date back to a remote past measured not by years, or even by centuries, but by geologic ages. With all his skill man has never been able to improve upon these methods of antiquity nor even to duplicate them. It is quite unlikely that man will ever be able to manufacture similar synthetic products equal in quality or approachable in price to those which Nature has thus provided. The reserves of these natural products are very great, however, so there is no apparent need for seeking any synthetic process for their manufacture.

Now wherein is the fundamental difference in the mode of attack in these two groups? Definite standards of performance must be established for any material, but for the natural product group, materials to fit the specifications may be obtained by selection only. In the group of synthetic or refined products selection is secondary to modification; selection of course may be made, but the fabricator has always before him the wide field of possible modification in composition or properties. The fabricator of natural rock products, however, has no such liberty—he must take his materials as he finds them and make the best of them. That last prosaic phrase, "make the best of them," contains the keynote of this whole topic. Confronted with the impossibility of changing the properties of his materials, the fabricator to get the best service from his products must use discrimination, he must pick and choose, and he must apply each product to the use for which it is best adapted.

An alloy steel is a fixed product, the proportions of its constituents are definitely established, and broadly speaking it may be duplicated anywhere in the world. The same is true to an even greater degree of all kinds of chemically pure salts, and a great variety of such synthetic materials have been classified and labelled. Warehouses are stocked with them, and buyers have the right to expect that shipments from widely separated sources and at different times will be interchangeable in use if not actually identical in quality. A sample of slate, granite or limestone, however, though it may be accurately defined in composition, hardness, color or porosity, may not be readily duplicated, and the determination of a sample from one locality is of only limited value in defining a similar rock in any other locality. Nature has demonstrated her ability to manufacture rocks of infinite variety; seldom has she produced rocks in two localities that are exactly alike. The earth is a great storehouse of such materials, the general types of which have been defined and classified. Some progress has been made even in determining the detailed characters of materials from specific localities, but on the whole the commodities in Nature's warehouse are unclassified. In this warehouse one finds no orderly shelf arrangement, and there is no card catalogue of what it contains. To complicate matters, new deposits are constantly being opened up, and as these new annexes and sub-basements are added to this great warehouse, there are more and more products that have to be tested and placed in their proper classification. The metallographers have before them the never-ending task of testing the composition and properties of an infinite variety of alloys that may be compounded; those who investigate natural products likewise have a never-ending task because of the infinite variety of products that Nature has already made.

For the group of natural products having fixed and unchangeable properties, the first task is to gain an intimate knowledge of chemical and mineralogical composition, impurities, structure, texture, hardness, durability and strength. The Society's Committee D-16 on Slate began at the very foundation. In its deliberations it soon discovered the somewhat disconcerting fact that we did not know what slate was. In order to discourage the use of inferior material it seemed desirable to confine the term "slate" to standard commercial products, and it was necessary to define accurately what we were talking about. With the aid of the best geological and mineralogical talent obtainable, it proceeded to write a definition that is now a standard of this Society.[1]

A clear definition of slate tends to remove misconceptions regarding its origin and character. Since, as nearly everyone knows, slate is derived from clay, a natural conclusion is that it consists largely of clay and that when exposed to severe weather conditions it will again decompose to a clay-like mass. But this conception is entirely wrong. An amateur poet recently wrote a verse on slate, one line of which reads: "Slate's a magic slab of pickled mud." Possibly "pickled" is not the best term to describe what the geologists call "regional metamorphism," but nevertheless it does imply some sort of transformation from the original mud. As revealed by the definition which Committee D-16 has drawn up, this transformation has been profound. Clay appears only as one of the minor constituents; in the better types of slate it amounts to not more than two or three per cent of the total mass. All the remaining original clay matter has been changed into mica, quartz, chlorite and other minerals which are exceedingly stable and enduring.

From an adequate definition as a starting point, a fund of valuable information has gradually been built up. Compilations have been made from existing literature representing extensive studies by the U. S. Bureau of Mines, by the U. S. Geological Survey, by colleges and by testing laboratories, and new information is constantly being added.

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[1] 1927 Book of A.S.T.M. Standards, Part II, p. 784.

This general knowledge relating to slate is but one short step toward our ultimate goal. The uses of slate are multiplying, and our purpose is to gain the fullest possible information on the relation of physical and chemical properties to each individual use. It has been found, for example, that a calcium content is undesirable where slate is exposed to sulfur fumes, and again that the value of slate for electrical use depends to quite an extent on a minimum content of carbon and moisture. These examples chosen out of many that have been discovered illustrate a fundamental principle which cannot be overemphasized—namely, that in dealing with these products which are used as Nature formed them, selection for use is of paramount importance. An electrical slate must be free from those materials that affect its conductivity; a roofing slate should have low absorption and under certain conditions a low calcium content; for use in sills and caps a slate must have strength; and for steps and walks it must resist abrasion. Slate has remarkable properties that adapt it peculiarly to many uses, but it is obvious that a high-grade electrical slate might make poor steps, and a satisfactory structural slate might fail as a roofing material. The accumulated knowledge which is gradually being built up is a step toward classifying the materials in Nature's slate warehouse and putting them on their respective shelves. I am confident that this knowledge will gradually enable the slate manufacturer to make a better selection, using for each purpose only those raw materials which careful tests reveal as having superior properties for the designated uses. Thus we may expect better and better service with fewer failures, not because of any improvement in the character of the raw materials, as is often the case in metals and alloys, but through intelligent selection and by diverting each particular type to the use for which it is best adapted.

When we consider the many uses of natural rock materials and the multitude of deposits now being quarried, it is evident that a vast work of testing and classification still remains to be done before the innumerable rock ledges can be assigned to the uses for which Nature has best endowed them.

In the preceding pages it has been pointed out that the first step in testing slate and similar natural products is to determine as completely as possible the composition, character and properties of the materials, and that the second step, which we have now reached, is to set up standards of performance for each individual use and to determine the properties of various slates in relation to these uses in order that manufacturers may make an intelligent selection, and that the user may have a definite measuring stick by which to determine the suitability of the product that he buys.

In undertaking the study of products in relation to specific use we are confronted with new problems, for more than one material may be concerned. Where several materials enter into the construction of a single product the final result is influenced by the qualities of each individual component. We are striving for better and better products, and our success depends to a large extent on the proper adaptation of one constituent to another. The addition of new cloth to an old garment has long been regarded as a failure. A perfect material used in association with a second material of unsuitable properties may give very unsatisfactory results.

The life of a slate roof, for example, depends upon the durability of both the nail and the slate; similarly, the quality of a flat roof made of slate slabs set in mastic depends upon the quality and consistency of the asphalt as well as upon the properties of the slate. A laundry tub may be made of the highest quality slate and fashioned with superior skill, but it is a failure if put together with an inferior cement that permits leakage. Has a committee on copper or metallic coatings given any attention to specifications for copper or coated nails to be used on slate roofs? Have any of the asphalt sub-committees considered specifications for a roofing mastic that will give the best possible service on flat slate roofs? Has any committee on lead or lead products given attention to the compounding of litharge with other ingredients for making a water-proof cement to be used with slate in the manufacture of laundry tubs or shower stalls where the joints are vulnerable points?

This interrelation of materials affects processes as well as products. The wire saw is now used extensively in slate quarrying, and it seems probable that the qualities of a wire which make for the highest efficiency in a guy rope or in a hoist cable may be entirely different from those of a wire used as a tool for sawing slate. In so far as I am informed, no wire rope committee has yet investigated the properties of a wire best fitted for abrasive use. Is there not a wide field also for manufacturers of other stone-working equipment to make a more intensive study of their tools and appliances in relation to the properties of the rock upon which they are to be employed?

I have cited a few examples which seem to me to show the need of closer coordination of committees of this Society as affecting slate products. How great this need may be in other branches of the mineral product industries I cannot say. No doubt a great deal of such interrelated work is now being done by the Society, but it seems to me that a new era has been reached where the excellent work of individual committees must be supplemented to a greater extent by coordination. Community of effort for the common good is a maxim of to-day. In this age, products and processes of greater and greater complexity are being employed, and modern demands of service are ever more exacting. Increasing refinements in use and the gradual upward trend of specifications emphasize the demand for broader and more comprehensive testing.

An investigation undertaken by Committee D-16 on Slate led to an ideal illustration of co-partnership in committee work. After the appointment of Sub-Committee V on Weathering Characteristics it was learned that similar studies were being made on concrete, brick, tile and building stone. It was realized that the combined resources of these various sub-committees would attain the most effective results, and a Joint Committee on Weathering was established.

In early human history the family unit was sufficient unto itself, but with the increasing refinements of civilization the interests of the individual reach out further and are more complicated. Each of us nowadays is unavoidably drawn into the affairs of his community, of his state, of his country and even of the world at large. The work of the Society is passing through a similar process of evolution. Each of its individual committees has performed noble service in determining fundamental data of inestimable importance relating to the properties and capabilities of its products. Considerable work has also been done in determining specifications for definite uses, but with the growing interdependence of materials the committees have an increasingly close relationship. In order to broaden their tests, committees are expanding the scope of their activities, and I am wondering if in some instances, even though the object is laudable, they are not usurping the functions of other committees better equipped to follow up studies in the overlapping fields. Fortunately the Society already has the machinery for organizing interlocking activities, a Committee E-9 on Correlation of Research. If my appraisal of future trends in the work of this Society is even approximately correct, the work of this committee should become of very great importance.

SOME FUNDAMENTAL DEFINITIONS OF RHEOLOGY

BY EUGENE C. BINGHAM[1]

SYNOPSIS

Two hundred and fifty years have passed since Hooke's work on elastic deformation and two hundred years since the work of Newton on the flow of fluids, yet the knowledge of the deformation and flow of matter in its various states scarcely yet constitutes a science, but to avoid lengthy circumlocutions in the future it may be pardonable to refer to this branch of knowledge as *rheology*. Elasticity is confused with plasticity by referring to plastic flow as "imperfection of elasticity". The use of the simple laws of fluid flow when dealing with the flow of solids involves the gratuitous assumption that all types of plastic flow are identical with the linear type of fluid flow. This adds to the confusion.

During recent years efforts have been made to distinguish some of the types of plastic flow and to discover the laws governing them. But with this work exact definitions become necessary and these in turn have a bearing upon our technical definitions, specifications and methods of testing and measurement. This paper suggests in a tentative manner some of the definitions which are involved. They have already been criticized by a large number of scientists and are offered at this time to stimulate still further criticism.

After so many years there naturally exists a great diversity of opinion, but with more adequate knowledge there exists the possibility of a clarification of our point of view, if the problems can be approached patiently.

INTRODUCTION

In 1678, roughly two hundred and fifty years ago, Robert Hooke announced his justly famous law of deformation, "*Ut tensio sic vis*" or the deformation of a body is proportional to the shearing stress producing it, or finally in the more elegant language of mathematics $s = eFr$.

In 1685, or over two hundred years ago, Isaac Newton announced in his *Principia* the fundamental law of fluid flow, namely, the rate of fluid deformation is proportional to the shearing stress or $v = \phi F r$. It is fitting that we here pay homage to these great events.

The stream of knowledge in regard to the nature of flow was fairly turbulent two centuries ago, but later it seemed to become viscid, no longer exemplifying that classical dictum of Heraclitus πᾶντα ῥεῖ,

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"everything flows". Progress required that stress which is sometimes supplied by experimental facts—and these were lacking—so it became necessary to await the arrival of that great experimenter, Poiseuille, who in 1842 opened the flood gates to a reservoir of knowledge, still unexhausted. To that source may be traced back the brilliant theoretical papers of Stokes and Maxwell published in 1849 and the years following, also perhaps the clear distinction by Maxwell between fluid and plastic deformation and certainly the appreciation by Graham of the importance of flow in colloid chemistry.

If we compare the state of our knowledge in regard to the movement of electrons with our knowledge of the movements of molecules, we find an interesting analogy. In 1828 the conceptions in regard to the definitions of electrical quantities were very vague, but in that year Ohm presented his paper giving fundamental definitions. With Faraday's brilliant series of investigations, which soon began, there rapidly developed the magnificent science of electricity. of obvious industrial importance. In spite of all early advantages, the flow of matter is still not understood and since it is not mysterious like electricity, it does not attract the attention of the curious. The properties are ill-defined and they are imperfectly measured if at all, and they are in no way organized into a systematic body of knowledge which can be called a science.

"Rheology":

The subject under discussion when designated as the fundamental definitions of the *science of the deformation of matter* is cumbersome. The same objection may be raised to the designation of *fluidity and plasticity*. If, like Alice in Wonderland, we can make terms mean what we want them to mean, probably consistency would be the favorite designation, but this word consistency is a jewel in some eyes that would not perhaps look with favor upon its being applied to *paste*. A large number of words have been suggested and John R. Crawford, late Professor of Latin at Lafayette College, inclined to the belief that "rheology" is the most appropriate designation. This term will be used for convenience in what follows.

Definitions:

The Technical Committee on Consistency, Plasticity, Etc., of the Society's Committee E-1 on Methods of Testing, has become aware of the difficulty of finding adequate definitions for the terms of rheology in the older literature. If it is true that problems once stated are half-solved, then our definitions may well have our earnest consideration. The terms length, shortness, tackiness, etc., when applied to flow might almost be described as part of the language of the underworld of science since these words are not recognized by exact definitions in the literature. Other terms like hardness, strength, rigidity and elasticity have two or more distinct significations. Many of the terms of rheology cannot be given a quantitative measure, and among such terms we can cite both consistency and plasticity. They are therefore only semi-respectable.

Since scholars have often pointed out the tendency that exists for words to become debased, it should be a particular pleasure to go wordslumming in the attempt to reclaim some of these words, many of which will be found to come of "the finest old Nordic stock." The definitions are set forth below:

Opportunities for Greater Precision of Measurement:

If the problems of rheology are to be solved, there is need not so much for more data as for data of far greater accuracy than is at hand at present. Edward Washburn, Editor of International Critical Tables, informs me that the data of rheology are particularly poor and my own study of the fluidity data confirms this impression.

Most of the troubles of the past have arisen from three sources: (*a*) There has been a lack of a uniform standard substance for use in calibration, (*b*) there has been a general disregard of simple but important corrections, and finally, (*c*) tests are not ordinarily made to ascertain whether the instrument in use is functioning properly or not.

Returning to the subject of the fluidity standard, some responsible groups such as this Society, the U. S. Bureau of Standards and the International Union of Pure and Applied Chemistry could adopt the fluidity of some substance or substances at a given set of conditions of temperature and pressure as standard. Since the fluidity of water at 20° C. and atmospheric pressure is almost exactly 100 rhes, it would make a suitable standard. In measuring a fluidity there is no more need of measuring the diameter of the capillary than, in getting the weight of an object, there is need for establishing the relation between the cubic decimeter and the kilogram; but it is necessary that we *agree* upon a standard, as is certainly not the case at present. We may agree upon it permanently as in the case of the kilogram weight just referred to, or change the standard slightly from time to time, as we do in the case of atomic weights; different and possibly incorrect standards would be eliminated in either case.

Teaching Rheological Technique:

An examination of the laboratory manuals of physics and physical chemistry both in this country and elsewhere affords an explanation of the inaccuracy of our data already referred to. Methods are prescribed which may lead to inaccuracies of over ten per cent, without a hint of the presence of these large inaccuracies or giving a method for detecting them or for correcting them. The use of a variable pressure method to determine the validity or invalidity, under different circumstances, of the fundamental law of viscous flow is an exercise of the greatest pedagogical interest as well as practical importance. But to ask a student to calculate his probable error amounting to tenths of one per cent when the concealed systematic error of the instrument supplied him amounts to several per cent is deserving of condemnation. Even so capable a scientist as Charles Waidner of the Bureau of Standards once expressed to the author the common belief that it is impossible to measure fluidity with an error of less than two or three per cent. It is therefore particularly desirable that students should be shown how simple it is to get many times that precision.

Applications:

The need for more exact measurements and the use of newer methods is most manifest in colloid chemistry and in its applications to metallurgy and physiology. With the break-down of Poiseuille's law there has emerged the whole subject of plastic flow pressing for attention.

It would be possible to name one industry after another to which rheology may contribute until there would be scarcely any left. For example, flow is of importance in book-binding, in printing and in pencil making. In the ceramic industry the importance of plasticity has longest been recognized, but unfortunately for some reason the flow itself has not been measured in a way to be of significance, but other things supposed to be related to the flow have been measured time without number, such as dye absorption, shrinkage, feel, etc., until the real character of the flow properties has been entirely lost sight of. Probably no one can maintain that he can calculate the plasticity from the dye absorption. Our measurements of deformation are in a condition which may properly be described as chaotic. It is desirable to determine the different forms of deformation, for example, viscous, plastic, elastic, etc., and the laws which govern them, after which, and only then, control tests may be rationally suggested. Many methods which are now regarded as more or less fixed as standard appear to measure deformation, although the result may be in terms of time, distance, temperature, apparent viscosity, concentration, etc. After the laws of deformation are understood it will be fair to inquire whether these properties are simply related to the fundamental properties of rheology. If the properties now being measured are not simple fundamental properties it may be better to devise new methods of measurement which will give a clearer indication of the properties which we desire to control.

DEFINITIONS

The definitions given below are offered to stimulate discussion and to elicit criticisms:

Consistency.—Consistency is that property of a material by which it resists permanent change of shape, and is defined by the complete force-flow relation. The term may be applied to either fluids or solids, for example, consistency of iron, clay slip, syrup or butter. (Latin, *con*, together, and *sisto*, stand.)

Fluid.—A substance which is continuously deformed under any finite shearing stress, so long as the stress is maintained. Specifically a gas or a liquid. (Latin, *fluo.*)

Fluidity.—The measure of the rate with which a fluid is continuously deformed by a shearing stress. The property of flowing freely. *Quantitative.*—The fluidity is measured by the relative velocity in centimeters per second imparted to either of two parallel planes one centimeter apart by a shearing stress of one dyne per square centimeter, the space between the planes being filled with the fluid. The unit of measurement is the rhe.

Viscosity.—The measure of the resistance to continuous deformation in a fluid. The property of fluids by which they resist continuous deformation, especially characteristic of thick liquids like molasses or pitch. The reciprocal of fluidity. *Quantitative.*—The viscosity is measured by the shearing stress in dynes per square centimeter required to move either of two parallel planes, one centimeter apart relative to each other, with a velocity of one centimeter per second, the space between the planes being filled with the fluid. It is measured in poises or preferably in centipoises, the centipoise being the approximate viscosity of water at 20° C. (1.005 cp.). (Latin, *viscum*, mistletoe.)

Viscid.—Having a high viscosity. Opposite of fluid.

Solid.—A solid, in contradistinction to a fluid, is a substance which will continuously resist deformation provided that the shearing stress is below a certain value, which is called the yield value. At shearing stresses below the yield value, the deformation consists of the temporary elastic deformation and the permanent pseudo-plastic deformation. (Latin, *solidus*, dense.)

Plasticity.—A solid is plastic when it can be readily molded into a desired form. The shearing stress must of course exceed the yield value, and liquids do not exhibit plasticity because they do not retain their shape and therefore cannot be molded advantageously. Plasticity is that property of a solid in virtue of which it not only resists permanent change of shape but it resists relatively more and more as the shearing stress is lowered. Plasticity may not be defined quantitatively because it is a complex proprety made up of yield value and mobility or their equivalents. The opposite of brittle or non-plastic. (Latin, *plasso*, form.)

Greaves-Walker prefers Hewitt Wilson's definition,[1] as follows:

> "Plasticity is that property which enables a material to be deformed continuously and permanently without rupture during the application of a force which exceeds the yield value of the material."

P. H. Bates feels that the definition is a little intricate and might be given more clearly and briefly as follows:

> "Plasticity is that property of the material by which, as the shearing stress is lowered, the material resists change of shape with a decreasing ratio of flow to force."

Yield Value.—The minimum shearing stress in dynes per square centimeter required to produce continuous deformation in a solid. One of the quantitative factors in plasticity.

Mobility.—The measure of the rate with which a solid is continuously deformed after the yield value has been exceeded. The second quantitative factor in plasticity. *Quantitative.*—The mobility is measured by the velocity in centimeters per second given to either of two planes one centimeter apart by a shearing stress of one dyne per square centimeter in excess of the yield value, the space between the planes being filled with the plastic material. Mobility is a more general term than "fluidity," the latter being the mobility when the yield value is zero. (Latin, *mobilis*, movable.)

Stiffness.—The measure of the resistance to continuous deformation in a solid in excess of the yield value. The reciprocal of mobility. Characteristic of glutenous substances such as glue, rubber, etc. *Quantitative.*—The stiffness is measured by the shearing stress, in dynes per square centimeter in excess of the yield value, required to shear either of two parallel planes, one centimeter apart, with a velocity of one centimeter per second, the space between being filled with the plastic material. Stiffness is a more general term than viscosity, the latter being the stiffness when the yield value is zero. (Anglo-Saxon, *stif.*)

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[1] "Ceramics: Clay Technology," McGraw-Hill Book Co., p. 55.

G. S. Haslam states that this term is so generally established in the rubber and metal fields for substances with a high elastic limit that it seems inadvisable to use it in the sense of the above definition. If this is admitted, what is the best word to use? Possibly thickness or immobility.

Elasticity.—When substances have been compressed they tend to recover their volume as soon as the compression is removed. All fluids have perfect elasticity of volume. Elasticity of shape refers to spontaneous return of a substance after deformation to its former shape. The term is used in two distinct senses. According to one a substance is highly elastic which is easily deformed and quickly recovers, that is, it depends upon the amount of the deformation or recovery. According to the second usage that substance is the most elastic which most completely recovers its shape after deformation. Elasticity should be measured according to the first definition, and the lack of perfect recovery used to calculate the plasticity.

Elasticity of volume is measured by the compression in cubic centimeters of one cubic centimeter of material by a pressure of one dyne per square centimeter. The elasticity of shape is measured by the displacement in centimeters given to either of two parallel planes one centimeter apart by a shearing stress of one dyne per square centimeter, the space between being filled with the elastic material. The reciprocal of rigidity.[1] (Greek, *elauno*, drive.)

Rigidity.—The property of bodies by which they resist an instantaneous change of shape. The reciprocal of elasticity. In physics a rigid body is assumed to be not only rigid but stiff as well. *Quantitative.*—The rigidity is measured by the shearing stress required to displace elastically either of two parallel planes one centimeter apart to the extent of one centimeter, the space between the planes being filled with the elastic material. (Latin, *rigidus.*)

Cohesion.—The force, molecular in origin and perhaps the resultant of the attractions and repulsions between the electrical charges, which causes the particles of a substance to attract each other and tend to stick together. (Latin, *con* and *haereo*, stick together.) Tensile strength and surface tension are related but not an exact measure.

Adhesion.—The force which causes the particles of unlike substances to attract each other and tend to stick together. (Latin, *ad* and *haereo*, stick to.) Methods have been proposed for measuring the adhesion between solids and liquids, that is, their wettability.

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[1] Haslam says that elasticity is not the reciprocal of rigidity.

Strength.—The maximum stress required to overcome the cohesion of a material. *Quantitative.*—A complex property made up of tensile strength and shearing strength. The force required to break a bar of unit cross-section under tension, that is, the tensile strength, depends not only upon the cohesion but also upon the consistency and therefore at least to some extent upon the rate of application of the load. Strength involves the idea of resistance to rupture, while hardness involves resistance to deformation, according to R. S. Archer. Distinction may be made between tensile strength and compressive strength.

Length.—Length is a quality possessed by a solid having a low yield value and a low mobility, ductile. A long substance has the characteristic of a very viscous liquid. It is capable of being drawn out into ropes, threads or fibers. Compare shortness, toughness and tackiness.

Shortness.—Shortness is a quality possessed by a solid which has a high yield value and high mobility, non-ductile, for example, butter. A short solid "necks down" rapidly and shows little tendency to be drawn out into threads. Emulsions are all short. The opposite of length. G. S. Haslam regards cohesion as an essential factor, and the mobility as low.

Hot Shortness.—When a material loses ductility or malleability at high temperatures so that it cannot be hot worked it is hot short or brittle. This state may be produced in a metal when one of the components becomes liquified while the remainder remains solid. As the yield value of the more fluid part is exceeded the flow takes place readily, as in an emulsion.

Cold Shortness.—A condition of brittleness at ordinary temperatures which may arise for example, by pouring a metal during solidification. Seams are produced which result in brittleness.

Toughness.—The ability to withstand large and sustained shearing stresses without yielding. Toughness involves low mobility (or fluidity) and adequate yield value. Automatically a certain cohesion is involved. See also tackiness, length and shortness. (Anglo-Saxon, *toh*, tough.)

Tackiness.—Stickiness. A quality possessed by a solid having a low yield value and high mobility, by means of which contact readily results in adhesion; for example, glue, varnish, printer's ink and gold size under working conditions are tacky or sticky substances. When a material dries out, sets up or gels, or hardens due to chemical or other change it loses tack or stickiness. Compare length, shortness and toughness. (Breton, *tach*, tack.)

Hardness.—A property applied to solids and very viscous liquids to indicate solidity and firmness in substance or outline. A hard substance does not readily receive an indentation or impression. It is unyielding to a bruising, cutting or penetrating instrument. In viscous liquids the hardness is most appropriately measured by the viscosity, but the hardness of solids must be measured by the two properties of yield value and mobility and there must be different kinds of hardness. Being a complex property, it is unfortunate that most of the methods in use attempt to express this property by a single numerical value. But it is at any rate recognized that the methods in use do not measure the same kinds of hardness.

In grading minerals the hardness is based upon the ability of one mineral to scratch another. This scratch hardness involves the factor of cohesion. The sclerescope hardness test depends upon the amount of rebound of a steel sphere dropped upon the surface to be tested and involves the factor of elasticity. Since very viscous liquids like pitch with complete absence of yield value and low tensile strength may yet possess a certain form of hardness, it seems reasonable to assume that a low mobility is essential and distinguishes hardness from toughness; since a tough substance may be deformed whereas a merely hard substance may or may not. Rigidity is sometimes regarded as a form of elastic hardness. (Anglo-Saxon, *heard.*)

Softness.—The opposite of hardness. A soft substance is easily deformed permanently without fracture. Contrast with brittleness. It is not identical with weakness; for example, a soft pencil lead may possess considerable strength. (Anglo-Saxon, *softe.*)

Pasty.—Having the consistency or appearance of paste. The latter use is not scientific as, for example, a pasty complexion. Applied to a rather short material. (Greek, *paste,* barley.)

Glutinous.—Having the consistency of glue, applied to a long, a stringy substance.

Livery.—Having the consistency or the appearance of liver. The latter use is unscientific. Applied to certain materials such as paints which sometimes develop an excessive yield value on standing.

Greasy.—Like grease or oil. Smooth, seemingly unctuous to the touch. Slippery. Talc and graphite have a greasy feel. The property seems to be caused by the layer of viscous fluid or soft solid particles, which forms the lubricating film between the two surfaces sliding over each other.

Lean.—Non-plastic, lacking the ingredient required to give the desired yield value; for example, pastry is lean when it lacks sufficient lard, that is shortening.

Fat.—A substance which is very plastic or capable of becoming so on the addition of a dispersing agent. Applied to clay for example.

Slippery.—Applied to a surface over which objects readily slip or slide. A smooth and well-lubricated surface is slippery. Ice is more slippery than glass, presumably due to the film of water acting as a lubricant. (Anglo-Saxon, *slipan.*)

Lubricant.—A substance used to reduce friction or making surfaces slippery. The lubricant tends to eliminate the effects of asperities of the surfaces, to prevent adhesion between the surfaces and to introduce a material, either solid or fluid, of sufficient consistency to maintain a film which can be easily sheared.

Body.—The appearance given by consistency, density, or opacity, of possessing much of the substantial or valuable ingredient; for example, wine has body due to color, silk due to its weight, lubricating oil due to viscosity and paint due to the use of a blown, that is, a bodied oil or of a soap as aluminum stearate known as false body. The bodied oil gives a low mobility and the false body results in a high yield value.

Solubility.—The conventional definition of solubility is the percentage of dissolved substance contained in a saturated solution at a given temperature. But the term cannot be given the conventional definition when it is applied to colloids for the following reason. Saturated solutions of colloids cannot in general be prepared, and the solutions are quite different from ordinary solutions in that they do not obey the laws of true solutions as regards osmotic pressure, vapor pressure, freezing point, etc. Colloidal solutions are not true solutions but fine dispersions, with the particles of the solute always of more than molecular magnitude. A good solvent is a medium which disperses a given colloid to give a material of low consistency. Thus acetone is a good dispersing agent or solvent for nitrocellulose whereas benzene is a non-dispersing agent or non-solvent. So colloidal solubility may be defined as the percentage concentration of colloid required to give a yield value when brought into solution at a given temperature. Generally speaking the colloidal solubility increases with the temperature.

Plasticizer.—A substance which lowers the consistency of a mixture. Some substances are of too high a consistency to act as colloidal solvents, even though they are able to disperse the colloids. Thus camphor converts nitrocotton into a plastic mass rather than a colloidal solution. It is therefore known as a plasticizer.

Melting Point.—The melting point is the transition point between solid and liquid phases. The melting point or the softening temperature of a colloid does not have the conventional meaning given above since it does not refer to a change of state but to a more or less rapid change in consistency. When a pure liquid like ethanol is cooled without crystallization, it becomes more and more viscid and finally vitreous, but it is quite arbitrary to speak of a definite solidifying temperature. With colloids, however, there appears to be a temperature at which the yield value appears and this may be regarded as the analogue of the melting point. The melting temperature and the solidifying temperature should be identical. In a colloidal solution the temperature of incipient melting and complete melting may of course be quite different.

Workability.—Certain substances change in consistency during the period of application or of manufacture; for example, plaster suffers an increase in consistency as it is being applied to the walls, due to water absorption. A material shows the property of workability which is readily manageable during the operations in question.

Thicken.—To increase the consistency of a material. Opposed to thin.

Inspissate.—To increase to a desired consistency, thicken. Opposed to temper. (Latin, *in* and *spissum*, thicken.)

Temper.—To reduce or bring to a desired consistency, plasticity, softness or tractability, as by adding solvent, dispersing agent, deflocculating agent, by stirring, mixing, kneading, pugging, grinding, etc. The hardness of steel is tempered by heating.

Cut.—The physical action by means of which a material is divided or severed. The action of cutting is partly plastic flow but also a breaking apart by the wedging or prying action of the knife or tool. In general the less the factor of plastic flow has in the process, the less will be the energy used up in the operation of cutting. This efficiency is often gained by lowering the angle of the wedge, by lubrication of the cutting edge, or by reducing the consistency of the material.

Brittleness.—The liability to break without appreciable deformation. Opposed to toughness. A material only exhibits brittleness as the shearing stress exceeds the cohesion of the material. A brittle material may be either solid or liquid but it has low cohesion and low mobility. Spun glass is brittle to the extent of being friable, that is, easily crumbled; but a spider's web is fragile and not brittle.

Ductility.—The capacity for deformation or extension without rupture by drawing or other process involving tension. A property of low yield value, low mobility with adequate cohesion.[1] (Latin, *duco*, lead.)

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[1] G. S. Haslam thinks a high mobility necessary.

Malleability.—The capacity for undergoing compressive deformation without rupture as in hammering or rolling. This is also characteristic of long solids. G. S. Haslam again dissents.

Since malleability, ductility, and similar properties depend upon the rate of shear, any attempt at the comparison of materials should be made under conditions which are truly comparable. (Latin, *maleus*, hammer.)

DISCUSSION

MR. W. H. HERSCHEL[1] (*by letter*).—An examination of the Appendix to the Report of Committee E-9 on Correlation of Research,[2] shows clearly the great need of agreement on terms involving consistency. One wonders why such a variety of terms is used in descriptions of the research projects. Under S 12, fluidity is mentioned, S 22 is an investigation of the cause of plasticity of lime, the viscosity and plasticity of paints are to be studied under S 30, and S 43 concerns the consistency of fluid cement-water mixtures.

It seems inadvisable to consider fluidity as the fundamental property and viscosity as merely its reciprocal. Possibly under certain circumstances, rarely if ever attained,[3] fluidities are additive, and fluidity may have a significance in regard to "association" and other matters intelligible only to chemists. Now, however, that a Society of Rheology has been organized, and it is desired to interest engineers, metallurgists and all others to whom the laws of flow are or should be matters of serious consideration, the time has come when it seems desirable to consider the point of view of the engineer.

To the engineer fluidity is only a negative quantity, an absence of viscosity, while viscosity, in the case of liquids, is the positive property which causes resistance to flow or loss of head. Similarly with concrete, the matter of interest is the force required in mixing or spreading the material in place, although concrete being a plastic material its force-flow relation is less simple than for a liquid and can not be expressed by any single numerical value.

The use of mobility and yield value as factors of plasticity appears undesirable because the former decreases with the concentration, while the latter increases. Plasticity is more readily comprehended when expressed by factors, all of which increase as the concentration or "thickness" of the material increases. As an example of such factors, the consistency of certain materials may be expressed by n, I and $F_0 = \frac{K\,d\,g}{4\,l}$, being approximately equal to the shearing stress at the walls of a capillary when flow begins, while n and I have previously been defined.[1] F_0 might be considered as an approximate value of the yield value as now defined by Bingham.

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[1] Associate Physicist, U. S. Bureau of Standards, Washington, D. C.

[2] *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part I, p. 538 (1929).

[3] E. C. Bingham, W. L. Hyden and G. R. Hood, *Industrial and Engineering Chemistry*, Vol. 16, pp. 25, 534 (1924).

If $n = 1$ and $F_0 = 0$, the material has a viscous consistency and its force-flow relation may be completely defined by the viscosity (or by its reciprocal, fluidity). If F_0 is negligible and n is greater than unity, the consistency of the material may be adequately identified by values of n and I. The best name for this class of materials is in doubt, but it is generally agreed that any material is a plastic solid if F_0 has an appreciable value.

The above factors of consistency have been applied only in cases where the rate of flow through capillary tubes is proportional to $(P-K)^n$, where P is the pressure in grams per square centimeter. Definitions of rheology, however, must be broad enough to apply not only to all types of flow through capillary tubes, but to flow in general, and Bingham's definitions of yield value and mobility must be judged on this basis.

The definition or determination of yield value does not need to be based on any assumption in regard to the force-flow relation, as shown by Green and Haslam.[2] It should be observed, however, that the tentative definition of yield value, now proposed, refers to actual starting conditions, and not to a hypothetical "yield shear value," f, calculated from a value of the pressure obtained by extrapolating an assumed straight portion of a force-flow graph.[3] Such a value of f does not at all agree with F_0.

The definition of mobility implies a linear force-flow relation for *all* plastic materials. As Bingham has pointed out:[4] "At first it was not recognized that there are quite distinct types of plastic flow, but we must certainly recognize three types and there may be others." After explaining that, in his opinion, suspensions are of the type of consistency which can be expressed by yield value and mobility, he goes on to say (page 221) that with polar colloids of the gelatin-in-water type, the flow is not proportional to the mobility, and "the difference seems to be not one of degree but to constitute a different class entirely."

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[1] Equation 6 of paper by W. H. Herschel and R. Bulkley on "Measurement of Consistency as Applied to Rubber-Benzene Solutions," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 628 (1926).

[2] Henry Green and G. S. Haslam, *Industrial and Engineering Chemistry*, Vol. 17, p. 726 (1925).

[3] E. C. Bingham, in discussion, *Proceedings*, Am. Soc. Testing Mats., Vol. XX, Part II, pp. 486–488 (1920); also W. H. Herschel and R. Bulkley, *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, pp. 630–633 (1926).

[4] E. C. Bingham, "Colloid Symposium Monograph," Vol. 5, pp. 219–228 (1928).

Even in the case of suspensions there is a question whether the proposed definition of mobility is tenable, since (except in one case which is open to considerable question), a linear force-flow relation has never been observed experimentally, when the tests were extended to sufficiently low rates of shear.

THE TRUE TINTING STRENGTH OF WHITE PIGMENTS

By G. W. Thompson[1]

Synopsis

This paper is an attempt to rationalize and to develop an equation for the relations between the brightness resulting from mixing white and black pigments and the composition of the mixture. Considering in a mixture the white pigment as if it had a brightness of 1 and the black pigment as if it had a brightness of 0, we designate the range from 1 to 0 so calculated as the proportional brightness, designated y.

Evidently the proportional brightness is dependent directly upon the proportion of white pigment present, and also the proportional brightness is dependent inversely upon the strength of the black pigment as well as the proportion of black pigment present. We find, however, that this is not all. The effectiveness of the black pigment in affecting brightness is much greater per unit of black substituted when the number of units of black substituted is small than when the number is large. By extrapolation it is shown that the effectiveness of the black as the proportional brightness approaches 1 is ten times the effectiveness of the black when the proportional brightness approaches 0. The effectiveness of the black varies as 10^y. Much has still to be done to bring this work to a practical conclusion, which means the development of satisfactory laboratory methods for determining brightness and preparing pigment mixtures.

When a black pigment is added to a white pigment and the mixture is rubbed up in oil the brightness of the product is intermediate —the brightness is less than that of the white pigment alone and greater than that of the black pigment alone. Let the brightness of these pigments in oil and mixtures in oil[2] be as shown in the following equation:

$$\frac{B_r - B_b}{B_w - B_b} = y \qquad (1)$$

in which B_w = brightness of white pigment alone,
B_b = brightness of black pigment alone, and
B_r = brightness of mixture.

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[1] Chief Chemist, National Lead Co., Brooklyn, N. Y.

[2] H. D. Bruce, "Tinting Strength of Pigments," U. S. Bureau of Standards *Journal of Research,* Vol. 1, No. 2, August, 1928, p. 125.

For convenience and for want of a better designation we shall call y the "proportional brightness."

$$y = 1 \text{ when } B_r = B_w$$
$$y = 0 \text{ when } B_r = B_b$$

It is obvious that y is affected directly by the proportion of white pigment in the mixture.

In any mixture of white and black pigments the amount of white pigment may be represented by W and the amount of black pigment by B. The proportion of white pigment present may be expressed by the ratio $\frac{W}{W + B}$. As W and B may represent any amounts it is well to have an expression which represents the amounts of white and black pigments calculated so that their sum equals one. Let w and b represent these amounts respectively and $w + b = 1$.

TABLE I.—RESULTS OF TESTS ON CERTAIN BLACK AND WHITE PIGMENTS.

y	b	r	r'
0.874	0.00083	172.	23.0
0.804	0.00144	169.3	26.7
0.643	0.00557	98.66	22.4
0.46	0.01645	70.0	24.3
0.35	0.0325	55.3	24.7
0.27	0.0608	40.7	21.8
0.195	0.0910	41.2	26.3
0.161	0.1185	38.72	26.7
0.146	0.1438	34.86	24.9
0.125	0.1705	34.05	25.5
0.023	0.6280	25.15	23.8
		Average	24.55

Then the relations between the amounts of the pigments used in any case may be represented by the following expressions:

$$\frac{W}{W + B} = \frac{w}{w + b} = \frac{1 - b}{1 - b + b} \text{..............} (2)$$

Pigments differ in strength. (In this paper where we use the word strength we refer to tinting strength). This is true of both white and black pigments. Differences in strength are due principally to differences in refractive index and fineness. The proportional difference in the strength of two pigments may be represented by the factor r (in any given test) which may be any figure above or below 1. The amount of a black needed to produce a certain proportional brightness depends upon the strength of the black. The stronger it is the less there is needed. In this discussion we shall consider that r refers to the strength of the black as compared with that of the white pigment. The expression rb indicates that as the strength of the black r increases, the amount of the black needed to produce a certain proportional brightness decreases. As b represents the amount of black used, which is a factor in affecting the proportional brightness y, rb represents the efficiency of the black in affecting proportional brightness y.

Let us now construct an equation which ought to express these general observations. Let us assume that:

$$y = \frac{W}{W + rB} = \frac{w}{w + rb} = \frac{1 - b}{1 - b + rb} \qquad (3)$$

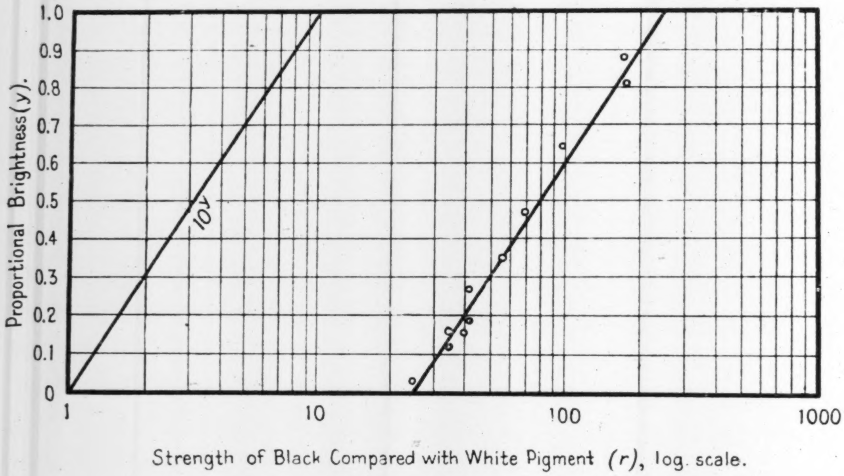

FIG. 1.—Relation of Proportional Brightness to Strength of Black Pigment.

Note that y is proportional to $w = 1 - b$ considered alone. Also that the greater the value of r the lower the value of y.

As a result of many rub-ups we have been forced to the conclusion that Eq. 3 is only true when r is a variable. The equation $y = \frac{1 - b}{1 - b + rb}$ therefore contains three variables: y, r, and b. Knowing the values for y and b we get the values for r. In Table I are given the results of eleven tests with a certain white pigment and a certain black pigment. There are shown the y and b and the calculated values for r. It is to be noted that r increases with y, indicating that r contains some factor varying functionally with y. Let us consider this factor as f, making $fr' = r$ and reconstruct our equation as follows:

$$y = \frac{1 - b}{1 - b + fr'b} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

By plotting y against $fr' = r$ on plain logarithmic paper (y plain and $fr' = r$ log scale) Fig. 1, we find that we get substantially a straight line slanting upward and away from the y axis. This introduces an interesting study into the significance of such lines on plain logarithmic paper. Considering the y axis plain and the x axis logarithmic, a straight slanting line running upward from the x axis may be considered the logarithmic graph of a series of products of two factors, one a constant and the other a variable. When $y = 0$ the variable factor reduces to 1 and the reading on the x axis gives the value of the constant.

Applying these principles we find the reading on the x axis to be 24.55 which is the value of the constant r' and the *true strength* of the black pigment with respect to the white pigment. Drawing a line parallel to the graph of r starting with $y = 0$ and $x = 1$ we find it crosses 10 on the log scale when $y = 1$. This line is then the logarithmic graph of the variable f and is expressable as 10^y. It is obvious that knowing the true strength of a black pigment compared with different white pigments the true strength of the white pigments with respect to each other may easily be calculated. We now get our final equation:

$$y = \frac{1 - b}{1 - b + 10^y r'b} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Apparently this equation is truly representative of the relations that obtain between the proportional brightness y of mixtures (in oil of a certain white pigment and a certain black pigment) and the amount of the black pigment used having a strength of $r' = 24.55$.

The values of r' (equals $\frac{r}{10^y}$) have been computed for each of the eleven tests and are shown in the last column of Table I, the average value being 24.55.

The experimental work so far done in developing this equation in the Research Laboratories of the National Lead Co. has been confined to three white pigments: white lead, titanox, and titanium oxide, and one black pigment especially prepared for strength investigations by reduction dry with a low-strength white pigment ground continuously for a week or more in a ball mill to get a uniform distribution of the black component. Further work is under way and a research associate is giving all his time to this subject at the U. S. Bureau of Standards. As this work progresses it is found that for the proper development of the work it will be necessary to devise practicable methods for securing a uniform distribution of the black particles among the white particles. It has been noted that the amount of oil used affects the reading somewhat. It seems probable that brightness readings will have to be made by either absolute methods or by standardized apparatus. With the three white pigments worked upon the equation given has been confirmed. It is not to be claimed, however, that this is true at or near the extreme ranges of proportional brightness where readings are difficult and perfect distribution of black particles questionable.

In Eq. 5, b represents an amount of black pigment substituted for white pigment. This substitution may be by weight or volume—that is, b may be weight or volume. There are four conceptions which may be followed in making strength determinations. These are:

1. Weight substitution.[1]
2. Volume substitution.[1]
3. Weight addition.[2]
4. Volume addition.[2]

According to these conceptions, b may be given sub-designations to distinguish them from each other, as b_1, b_2, b_3, and b_4, respectively. The values may be converted b_1 to b_2 and b_3 to b_4 by multiplication by the suitable specific gravity factor. The value b_1 may also be converted to b_3 and b_2 to b_4 as follows:

$$b_3 = \frac{b_1}{1 - b_1} \text{ and } b_1 = \frac{b_3}{1 + b_3}$$

Let us divide both numerator and denominator in the second member of Eq. 5.

$$y = \frac{\dfrac{1 - b_1}{1 - b_1}}{\dfrac{1 - b_1 + 10^y r' b_1}{1 - b_1}} = \frac{1}{1 + 10^y r' b_3} \quad \text{............} (6)$$

In Eq. 5 when $y = 0$, $b_1 = 1$.

In Eq. 6 when $y = 0$, $b_3 = \propto$.

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[1] In these two conceptions the two weights and the two volumes of white and black pigments are assumed to add up to 1.

[2] In these two conceptions the white pigment is considered as having a weight or volume of one and the black pigment added to have a variable weight or volume.

DISCUSSION

Messrs. G. S. Haslam[1] and G. F. A. Stutz[1] (*presented in written form*).—Mr. Thompson is undertaking a very commendable piece of work in attempting "to rationalize and to develop an equation for the relation between the brightness resulting from mixing white and black pigments and the composition of the mixture." The work which Mr. Thompson has reported and that which is now being done at the Bureau of Standards should be very helpful, particularly in view of the fact that the tinting strength test at the present time is of questionable value and has been discontinued as a test in the standards of this Society.

Two questions immediately come to mind from Mr. Thompson's relationship established between the quantities y and r. First, is the conclusion correct that, with a given black, all white pigments will give a curve with the same slope? Second, is the interpretation of the intercept on the r axis, r', justified?

If the slope of the curve is the same for all pigments then the tinting strength test, in its present form, should be entirely satisfactory except that Mr. Thompson's use of the intercept on the r axis probably more nearly approaches the "true" strength as he calls it, since it entirely eliminates any brightness effect.

However, from data obtained in our own laboratory and from published data we are led to believe that it is not possible to draw a straight line such as Mr. Thompson has in every case. We have plotted in the accompanying Fig. 1, according to Mr. Thompson's method, the data given by Bruce in his paper on "Tinting Strength of Pigments,"[2] which includes seven different white pigments, three of which Mr. Thompson has used in his paper. It will be noted that there is a marked departure from a straight line. The curve at each end should turn off very markedly from a straight line since the equation demands that as y approaches 1, r approaches 0, although at $y = 1$, r is an indeterminant. At $y = 0$, r is also an indeterminant, but it would seem that all curves should approach the same point on the r axis.

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[1] New Jersey Zinc Co., Palmerton, Pa., Research Division.

[2] H. D. Bruce, "Tinting Strength of Pigments," U. S. Bureau of Standards *Journal of Research*, Vol. 1, No. 2, 1928, p. 125.

The black which Mr. Thompson has used is not a pure black, but one doped with a white, and therefore has an appreciable brightness. In this case, the data he has obtained would involve only the middle range of brightnesses and so probably account for the straight line obtained.

From the data we have plotted it is of course evident that in a very general way it would be possible to draw a straight line for each pigment and that these straight lines would closely parallel one another. However, it does not seem sound to use the intercepts of such lines on the *r* axis and call them a measure of the true strength of the black relative to the white when the curve is drawn from points of intermediate brightness only. It seems to us that it would be more satisfactory to obtain the whole curve from $y = 1$ to $y = 0$ experimentally using a full-strength black and then draw the conclusions from this curve. Under such conditions we believe that the intercept on the *r* axis would lose its significance. It would then be necessary to use the value of *r* at some constant value of y, such as $y = 0.1$ or $y = 0.2$. This would simply be returning to the difficulty that Mr. Thompson is now endeavoring to overcome, and we are forced to return to the original unsatisfactory state of affairs in the tinting strength test.

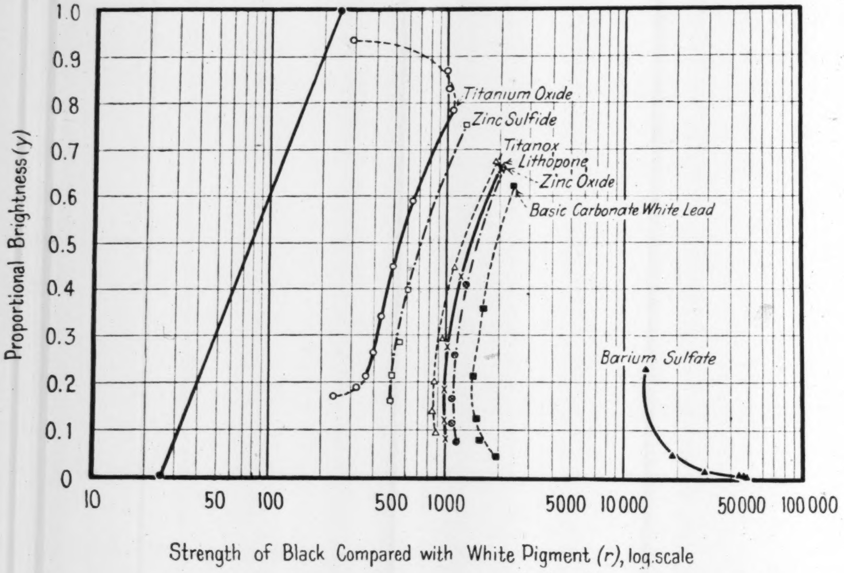

FIG. 1.—Bruce's Data Plotted According to Thompson's Method. Curve at extreme left is Thompson's data.

It is to be hoped that the further work being done on the subject will indicate the way in which Mr. Thompson's method may be modified to give results that will truly evaluate the tinting strength test.

MR. DEANE B. JUDD[1] (*presented in written form*).—I find that my objections to the text of this paper could all be met by revision of a single paragraph, the second paragraph on page 927, which deals with the "true strength of the black pigment with respect to the white pigment" and also with the "true strength of the white pigments with respect to each other." This paragraph implies that a single number may be assigned any black or white pigment (a number called the "true strength") which shall completely characterize its power to alter the brightness of any pigment mixture. Now the problem of pigment mixture is an admittedly complex one. Mr. Thompson implies that many factors are involved when he says (page 925), "Differences in strength are due principally to differences in refractive index and fineness." Other factors that are more or less effective are crystal structure, spectral transmission, and frequency distribution according to shape and size of pigment particle. Now if the effect of all these factors may be combined into a single number (such as the "true strength" proposed by Mr. Thompson) it must be true that each of these factors is in effect not independent of the others; it must be true, for example, that a low refractive index compensates perfectly at all concentrations for a high degree of fineness; that is, a certain small change in refractive index must cause exactly the same changes in brightness at all concentrations that are caused by a change in particle size; and it must be true that a mixture of large and small particles behaves when mixed with a third pigment exactly as a group of particles of a constant intermediate size behaves; and so on throughout the entire list of possible factors. This sort of dependence cannot reasonably be anticipated even for a single pair of the factors involved, let alone for all combinations of the many factors. What we can hope for is that the important factors are mutually dependent in this way to a degree of approximation which justifies for practical purposes the simplified plan of combining all the factors into a single variable. Before suggesting that a single variable is sufficient for practical purposes, a large number of pigment pairs should be tried out; I believe that Mr. Thompson does not present sufficient evidence to warrant this conclusion. He presents data on a single pair of pigments, and he remarks further that some confirmation has been obtained with three white pigments. But even if a single variable proved to be sufficient for a large number of pigments I believe that to call this single approximate all-embracing variable the "true strength" of the pigment is misleading. The *true* strength of a pigment cannot possibly be defined for all concentrations by a single number; it is a function of many variables.

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[1] Associate Physicist, U. S. Bureau of Standards, Washington, D. C.

I have found the method of attack on the problem used by Mr. Thompson an ingenious and stimulating one. It has led me to derive an independent empiric solution, which will be discussed later, and which may perhaps supplement the one discovered by Mr. Thomspon. I believe, however, that the following suggested changes would result in a more intelligible account of this method:

His Eq. 3 is presented purely as an assumption. With certain not too serious restrictions, this relation may be derived from *a priori* ground. It is an extension of the law that the brightness of a surface made up of spots randomly distributed but not resolved is the area-weighted mean of the brightnesses of the spots. According to this derivation the quantity r is the ratio of the proportion of black pigment on the surface to the proportion of black by weight (or by volume by means of a relation for b_2 as indicated on page 928.) I believe such a derivation would aid in the rationalization which Mr. Thompson has undertaken.

The quantity, r, "refers to the strength of the black as compared with that of the white pigment." But we read that the constant r' gives the "*true strength* of the black pigment with respect to the white pigment." It seems, then, since r is different from r' (see Table I of the paper), that we must logically think of it as the "untrue strength" or the "false strength" or the "pseudo-strength" or the "apparent strength." I believe this difficulty has arisen because both r and r' have been given names which do not fit their properties. My objections to naming r' the "true strength" have already been given. If the derivation of Eq. 3 be given as suggested above, the very definite meaning of the quantity r could be stated. As for r', that quantity is merely the value of r for zero white pigment ($w = 0$, $b = 1$) or for zero proportional brightness ($y = 0$).

It will be noted that Eq. 5 is not explicitly written; that is, y appears on both sides of the equation. It is possible, however, to solve for b explicitly:

$$b = \frac{1 - y}{1 - y + 10^y r' y}$$

In this form it is plainly evident that we may compute by Mr. Thompson's empiric relation the proportion of black pigment, b, required to produce a given value of proportional brightness, y. In the form given by Mr. Thompson it is only clear that y cannot be conveniently computed from a given value of b.

The importance of uniform distribution of the black particles among the white particles is emphasized on page 928. I think that these remarks might well be supplemented by mention of the importance of the dispersion of the particles. Or, if Mr. Thompson means to include "dispersion of the particles" when he says "uniform distribution of the black particles" such an intention should be made clear.

My concluding remarks have to deal with the substitute empiric formula which I have derived by Mr. Thompson's method of attack. He has found empirically that the relation between r and y for a certain pair of pigments is $r = r'10^y$. I wish to emphasize that there is nothing fundamental about this relation although as an empiric relation it is beautifully simple. There is no reason to suppose that another pair of pigments would obey this law; they might require the relation $r = r'n^y$, where $n \neq 10$; or they might require the law $r = r'n^{(y+m)}$, where m and n are arbitrary constants, or they might not even fall readily into an exponential law. I have found empirically a relation connecting r with b to a similar degree of approximation:

$$r = \frac{0.700}{b + 0.004} + 32.0,$$

where 0.700, 0.004 and 32.0 are arbitrary constants adjusted (though not with great care) to fit Mr. Thompson's data. From Eq. 3 we may then write:

$$y = \frac{1 - b}{1 - b + \left(\dfrac{0.700}{b + 0.004} + 32.0\right) b}$$

It will be noted that the proportional brightness, y, may be computed according to this relation for any given value of b. Since this relation gives very closely the same numerical results as Eq. 5, it might be useful in carrying out computations of y for which Eq. 5 is ill suited. I say "might be useful" because I believe both this relation and Eq. 5 are based on too few and too unreliable data to be of more than very limited use. Mr. Thompson is very careful to point out that the constant 24.55 refers to a certain pair of pigments only. He says too that Eq. 5 refers to this certain pair of pigments only. I think he ought also to mention specifically that it refers only to the precise method (which he does not describe) used for the mixing of the pigments (though, to be sure, he implies certain of these difficulties at the top of page 928). Then, I believe, the necessity for the thorough-going revision (already recommended) of the second paragraph on page 927 which deals with "true strength" will be quite evident.

THE EFFECT OF STORAGE ON THE PROPERTIES OF TURPENTINE

By W. B. Burnett[1] and H. K. Salzberg[1]

Synopsis

During the period August, 1927, to February, 1929, tests have been run at regular intervals on samples of gum turpentine from the storage tanks of the 2 in 1 Shinola-Bixby Corporation located in Indianapolis, Ind. The tests applied have been the Standard A.S.T.M. tests for distillation range, specific gravity and refractive index; the German specification tests for bromine number and evaporation residue; also the acid number. Tests on tanks in storage for 15 to 18 months (actual age 18 to 21 months or more) indicate that turpentine in storage for that time is entirely satisfactory for general commercial use as embodied by the A.S.T.M. Standard Specifications for Gum Spirits of Turpentine and Steam-Distilled Wood Turpentine (D 13 - 26).[2] However, the effects of storage are noticeable, and at the end of this period the turpentine does not meet the German requirements in regard to bromine number and evaporation residue.

It is common belief that turpentine may be held in storage for a year or possibly longer without the quality being seriously affected, provided it is kept in completely filled containers and unnecessary exposure to the air is avoided. Schorger[3] has studied the effect of storage on turpentine contained in an iron drum and in a wooden barrel, and has found very slight changes in boiling point, specific gravity and refractive index. He concludes that turpentine stored with reasonable care for a period of one year will show little or no change in physical properties. Veitch and Grotlisch[4] give certain precautions to be observed in storing turpentine and state that it may be stored for periods of from 12 to 18 months without any unusual changes resulting.

Since there was little definite information concerning bulk storage and since it was thought desirable to study the effect of storage upon certain properties of turpentine not studied by previous workers, the authors in August, 1927, started a series of tests in cooperation with the 2 in 1 Shinola-Bixby Corporation on samples of gum turpentine taken from their storage tanks in Indianapolis, Ind.

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[1] Pine Institute of America Fellowship, Mellon Institute, Pittsburgh, Pa.

[2] 1927 Book of A.S.T.M. Standards, Part II, p. 236.

[3] A. W. Schorger, "Study of Authentic Samples of Gum Turpentine," *Journal of Industrial and Engineering Chemistry*, Vol. 6, p. 541 (1914).

[4] U. S. Dept. of Agriculture *Bulletin No. 898* (1921).

These tanks are of the horizontal type with a capacity of 24,000 gal. each, being approximately 10 ft. in diameter and 40 ft. long. They are set with a 4-in. pitch to insure drainage and are painted externally with a battleship gray paint. Expansion is provided for by filling to a depth of $9\frac{1}{2}$ ft., and in the case of tanks filled during the winter months a slightly greater allowance is made. All tanks are fitted with check valves to relieve any inner pressure. Before being filled they are thoroughly cleaned and given two coats of garnet shellac.

Sludge is pumped from the tanks each month by means of a $\frac{1}{2}$-in. pipe extending into a $2\frac{1}{2}$-in. drain well at the lower end of the tank. The greatest amount of water and sludge is obtained on the first pumping when it amounts to 2 to 8 gal. At subsequent pumpings from $\frac{1}{2}$ to 1 gal. is usually obtained. When the tanks are emptied a layer of sludge $\frac{3}{8}$ to $\frac{1}{2}$ in. deep is found on the bottom of the tank.

Samples for analysis were obtained by means of a liquid thief of 24-oz. capacity. Two samples from the top, two from the bottom and four from the middle of each tank were mixed and the samples of this mixture shipped in tightly-corked tin cans to Mellon Institute for examination. The examinations were carried out at intervals of three or four months using the standard A.S.T.M. methods and apparatus[1] except that the sulfuric acid polymerization test and the color determinations were omitted. The color was always better than "Standard." For convenience, the temperature at which 90 per cent of the turpentine had distilled was determined rather than the percentage that had distilled at 170° C. For the determination of the bromine number and the evaporation residue, the standard German methods (Reichsausschuss für Lieferbedingungen)[2] were used. The acid number was determined in the usual manner.

A complete record was obtained on a total of seven storage tanks for a period of 15 to 18 months during which time all of the tanks remained entirely filled (except for the expansion space as stated above) and were not disturbed except for the monthly pumping of sludge and for sampling. For the sake of brevity, only representative determinations on samples from tanks Nos. 7 and 15 are given in

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[1] A.S.T.M. Standard Specifications for Gum Spirits of Turpentine and Steam-Distilled Wood Turpentine (D 13 – 26), 1927 Book of A.S.T.M. Standards, Part II, p. 236.

[2] *Chemiker-Zeitung*, Vol. 32, pp. 2250–2253 (1927); Pine Inst. America ***Abstracts***, Vol. 1, pp. 6–11 (1927); also H. Wolff, "Die Lösungsmittel der Fette, Öle, Wachse und Harze," pp. 66–85, Wissenschaftliche Verlagsgesellschaft, G.m.b.H., Stuttgart (1927).

Table I in which there is included also the corresponding A.S.T.M. and German specifications for purposes of comparison. These data are representative likewise of the data obtained for the remaining five tanks whose history was followed over a period of 15 months. When the time consumed in production, shipping and storage before the tests were started is considered the turpentine at the conclusion of the tests had an age of at least 18 to 21 months. Tank No. 7 which contained two tank cars of turpentine from Florida and one from Mississippi was filled in June, 1927, and therefore passed through two summer and two winter cycles of storage. Tank No. 15 which contained only turpentine from Florida was filled in September, 1927, and passed through two winter and one summer cycle of storage.

TABLE I.—PROPERTIES OF TURPENTINE AS AFFECTED BY STORAGE AND THEIR COMPARISON WITH THE COMMONLY ACCEPTED SPECIFICATIONS.

		Initial Boiling Point at 760 mm., deg. Cent.	75 per cent Distilled at 760 mm., deg. Cent.	90 per cent Distilled at 760 mm., deg. Cent.	95 per cent Distilled at 760 mm., deg. Cent.	Specific Gravity, $\frac{15.5^\circ}{15.5^\circ}$ C.	Refractive Index at 20° C.	Evaporation Residue, per cent	Bromine Number	Acid Number
29 tanks, average		157.7	161.2	163.8	167.2	0.8676	1.4702	0.26	210.4	0.34
29 tanks, minimum		156.5	.....	162.7	165.5	0.8656	1.4692	0.07	205.9	0.20
29 tanks, maximum		159.0	.....	165.0	169.5	0.8735	1.4712	0.64	216.4	0.55
A.S.T.M.,[1] minimum		150.0	.....	.....	.....	0.8600	1.4650	....	.....	....
A.S.T.M., maximum		160.0	.....	170.0	.....	0.8750	1.4780	....	.....	....
German, minimum		152.0 (156.5)[a]	.....	.....	.....	0.8550	1.4670	....	210.0	....
German, maximum		156.0 (160.7)	162.0 (167.0)	.....	.....	0.8720	1.4780	0.50	.....	....
Tank No. 7	at start	157.5	161.5	164.5	169.5	0.8680	1.4703	0.41	210.1	0.31
	after 7 months	157.5	.....	165.7	.....	0.8712	1.4710	0.80	210.1	0.32
	after 14 months	158.3	.....	163.9	167.7	0.8699	1.4712	0.76	208.8	0.29
	after 18 months	156.9	161.6	165.7	173.9	0.8696	1.4708	0.94	204.5	0.41
Tank No. 15	at start	159.0	161.7	164.4	168.0	0.8676	1.4710	0.21	209.6	0.27
	after 7 months	158.6	.....	164.8	169.6	0.8683	1.4709	0.40	206.5	0.31
	after 15 months	158.3	161.8	164.9	169.9	0.8678	1.4708	0.53	203.0	0.27

[1] A.S.T.M. Standard Specifications for Gum Spirits of Turpentine and Steam-Distilled Wood Turpentine (D 13 – 26), 1927 Book of A.S.T.M. Standards, Part II, p. 236.

[a] The German thermometer is a different style from the A.S.T.M. thermometer. The values given in parenthesis are thought to be comparable to the corresponding A.S.T.M. values and have been arrived at by applying the stem corrections suggested by Wiebe, *Petroleum Zeitschrift*, Vol. 7, p. 1304 (1912).

As a matter of interest in regard to the properties of gum turpentine now being produced there are included in Table I, three headings marked "29 tanks." The data given under these headings were obtained by a study of 710,000 gal. or 85 tank cars of turpentine unloaded during 1927 and 1928. They represent the initial determinations on 29 storage tanks, 27 filled to capacity and the other 2 filled to two-thirds capacity. The turpentine in these tanks had been in storage on an average of two months before it was sampled and tested in the manner previously described but these data can be taken as representing the properties of fresh commercial gum turpentine. Its origin by tank cars was: Florida, 63; Mississippi, 10; Alabama, 9; and Georgia, 3.

The most marked changes noted in the turpentine during storage were an increase in the amount of high boiling material present as shown by an increase in the temperature at which 95 per cent distilled, an increase in the amount of residue obtained upon evaporation and a lowering of the bromine number. As regards the latter two constants the turpentine after storage does not meet the German specifications.

This series of tests has corroborated the findings and opinions of previous workers, and in the opinion of the authors turpentine may be held in bulk storage in a temperate climate for at least two years and still be of a quality entirely satisfactory for general commercial use as embodied by the A.S.T.M. Standard Specifications for Gum Spirits of Turpentine and Steam-Distilled Wood Turpentine (D 13 - 26), provided recognized good practice in storage is observed. Unquestionably there will be slight changes and for some few special uses the quality might be considered to be affected; however, regarding the use of turpentine as a thinner or solvent, it is still a moot question as to whether an increase in high boiling residues is harmful or desirable.

Acknowledgment.—The authors wish to acknowledge the assistance of Mr. E. T. Marceau of the Gold Dust Corporation, who first brought this question to their attention and who supplied all samples. The authors also wish to make acknowledgment of the assistance of Mr. I. H. Odell, Jr., former Industrial Fellow of Mellon Institute, in the earlier stages of this investigation.

DISCUSSION

MR. C. H. HALL.[1]—In connection with the question of the storage of turpentine, two or three years ago we received a tank car of turpentine that had evidently been in storage from the previous season, which contained approximately 6 per cent of residual matter. That material was of a resinous nature. The oxidation process of turpentine tends to leave a gummy deposit, and it is, according to our opinion, undesirable. I think the general impression is that such resin is undesirable.

MR. W. B. BURNETT.[2]—What was the nature of the resin?

MR. HALL.—I do not know, but it tends to make varnish gummy and soft. I have observed the same thing with old finishing varnish left partly exposed in the can. After a few months' time when the material becomes old, you will find that it does not dry, and in all the experience I have ever had with turpentine when you develop that gummy residue it is undesirable.

I am rather surprised to hear that one can store turpentine for a period of three or four years without any effect, because I doubt if this turpentine was over a year old. I do not know the conditions under which it was stored but I know it was stored in such a manner that it developed some 5 to 6 per cent of residue, and it seemed to me to be an undesirable thing to incorporate in finishing varnish. I believe if one were to take all those turpentines and redistill them you would get all the original properties of turpentine. At least, so it seems to be in the laboratory—redistilling seems to bring those properties back, and a certain amount of residue remains.

MR. F. P. VEITCH.[3]—I should like to ask what Mr. Hall means by residue.

MR. HALL.—In the ordinary turpentine, on steam distilling, there is a very small residue.

MR. VEITCH.—What was the amount of residue in that turpentine?

MR. HALL.—I do not know. This was a resinous gum which was in solution in the turpentine as an oxidation process.

MR. VEITCH.—At what temperature of distillation—above 170°?

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[2] Pine Institute of America Fellowship, Mellon Institute, Pittsburgh, Pa.

[3] Chemist in Charge, Industrial-Farm Products Division, U. S. Bureau of Chemistry and Soils, Washington, D. C.

MR. HALL.—I do not know the process by which it was determined, but the fact of the matter was that if one put it on a piece of paper and evaporated it one would get a tacky, gummy residue left on the paper. Just exactly how the quantity was determined I cannot tell you, whether by steam distillation or by dry distillation, but the fact of the matter was it was different from ordinary fresh turpentine, as we ordinarily receive it.

MR. VEITCH.—It is very rare that gum spirits of turpentine or any other well-made turpentine with which I am familiar, in proper storage over a period of a year or possibly two years, will give any large residue of the character mentioned by Mr. Hall. There is in all turpentine a portion which does not distill below 170° C. This portion or residue may be as much as 10 per cent, but turpentine that contains 10 per cent of residue not distilling below 170° C. will usually evaporate satisfactorily and not leave more than 1 to 3 per cent of sticky, resinous residue. We have also examined samples taken from the same tanks from which the samples discussed by the authors were taken, and in the year and a half, I think it is, that they stood, the residue on evaporation, that is the residue remaining after evaporating on the steam bath, 10 cc. from a flat-bottom petri dish 90 by 10 mm., was less than 1 per cent in all cases except one, and in that case the turpentine when first put in the storage tank gave a residue of $1\frac{1}{4}$ per cent. It had not changed materially during storage for a year and a half.

MR. HALL.—I do not know anything definite about the history of this material or the storage conditions. The storage conditions, I judge, have a great influence. These we knew nothing about, except the assurance on the part of the seller that the material had not been contaminated. Any residue in this turpentine may have been due to the fact that it had been carried over from the previous season, or the season before. The fact that they had produced pure turpentine seemed sufficient reason on the part of the seller why it should be taken, regardless of the fact that it contained material that had formed since it had been produced, which made it different from what we felt spirits of turpentine ought to be.

MR. BURNETT.—I have some information that may be of interest to Mr. Hall. Recently there was submitted to us a sample of residues from turpentine that was placed in storage in the South a number of years ago for purposes of speculation. As you know, the market dropped and this turpentine was continued in storage. No special precautions were observed and recently when efforts were made to turn it back into the market it was found to have gone badly off color.

It was cleared up by distillation from an ordinary turpentine still. In this operation there was obtained about 3 per cent of residues of which a sample was submitted and we were told that it probably contained rosin, concentrated from the large volume of turpentine distilled. Mr. Hall also mentioned the rosin occurring in turpentine residues. However, such was not the case with this sample since it was found to have an acid number of less than 10 and a saponification number of about 30, both of which preclude the presence of any appreciable amounts of rosin. It appeared to consist for the most part of the polymerization and oxidation products of the terpenes that occur in turpentine.

Incidentally this material dries to a hard varnish type of film rather than the more gelatinous type of film obtained from ordinary linseed oil. The incorporation of a drier hastens the drying time materially.

I thought that might be of interest in this case because the residue was found to be a polymerization product and not rosin and I imagine that the conditions of storage were very similar to those mentioned by Mr. Hall—partially filled tanks, etc.

Mr. Hall.—I understand that this was not resin in the sense of ordinary rosin.

Incidentally, in that connection, some twenty-five years ago I did some work on turpentine in connection with the heat rise of sulfuric acid, and I developed a method for analyzing it, and in asking some other people to confirm the method—I remember I asked Mr. Hooker, who at that time was chemist for the Heaton and Milligan Co. to check it and he observed the fact that whereas the process gave a very definite temperature rise and very definite check on fresh turpentine, when old turpentine which had a gummy residue was used the rise became very excessive. When redistilled with steam the results became identical to what they had been with the standard, which very definitely pointed to the fact that this residue was not ordinary rosin, because rosin, I believe, under those circumstances would give little or no rise.

The fact of the matter is that the oxidation products in turpentine give a gummy, tacky product, because of the fact, as I have stated previously, finishing varnish that has been partially exposed in a can and which dried perfectly normally before, loses this property and one can get the peculiar odor of old turpentine. It may be that other reactions take place. Of course, one has oil in combination with dryers, and this may in some way be involved, but I feel very sure that the major cause is the gummy oxidation products caused by the slow action of oxygen on turpentine.

MR. P. H. WALKER[1].—Did you ever observe that effect when using mineral spirits?

MR. HALL.—I do not know whether the type of varnish I have in mind was ever made with mineral spirits. As a rule, the type of varnish I have in mind would be linseed oil, Kauri gum varnish, which is generally made with turpentine. There may be others who have had experience with that.

MR. VEITCH.—May I ask Mr. Hall if he actually used the turpentine in making varnish and found it unsatisfactory?

MR. HALL.—No, we did not. We now specify on all orders the amount of residue which we will accept.

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[1] Chemist, U. S. Bureau of Standards, Washington, D. C.

THE EFFECT OF THE MIXING TEMPERATURE ON THE PHYSICAL PROPERTIES OF AN OIL ASPHALT IN A SHEET ASPHALT MIXTURE

By John H. Bateman[1] and H. L. Lehmann[2]

Synopsis

This paper describes an investigation which was undertaken at Louisiana State University to determine the effect of the mixing temperature on the physical properties of an oil asphalt used in a sheet asphalt mixture. The effect of mixing asphalt with mineral aggregate in a heated condition causes changes in the physical properties of the asphalt; the penetration and ductility are lowered and the softening point is increased. The extent of these changes is affected by the temperature of the heated materials at the time of mixing.

Introduction

A previous paper[3] before this Society described a method of recovering for examination the asphalt from asphaltic paving mixtures, in which the complete removal of the solvent used for extraction was accomplished. That paper stressed the value of the method in determining the effect of various factors on the physical properties of asphalt when used in paving mixtures.

The present paper describes an investigation which was made for the purpose of determining the effect of the mixing temperature on one grade of asphalt. The original plan contemplated a study embracing asphalts from several sources and a wide range of consistency (as represented by the penetration test). Such a program would require a great deal of work and would cover a long period of time, and the original plan has been delayed because of lack of time. The results of the work on a single asphalt product were of sufficient significance to make their publication desirable.

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[2] Chemist, Louisiana Highway Commission, Baton Rouge, La.

[3] J. H. Bateman and Charles Delp, "The Recovery and Examination of the Asphalt in Asphaltic Paving Mixtures," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 465 (1927).

OUTLINE OF INVESTIGATION

The investigation embraced a study of the effect of the mixing temperature on the following properties of a single grade of oil asphalt:

1. Penetration at 32° F.;
2. Penetration at 77° F.;
3. Penetration at 115° F.;
4. Softening point; and
5. Ductility at 77° F.

Mixtures were prepared at the following temperatures: 250° F., 300° F., 350° F., 400° F., 450° F. and 500° F.

The asphalt was extracted from each mixture with carbon disulfide and the carbon disulfide was removed from the asphalt by distillation. The recovered asphalt was then tested and the results of the tests were compared with the original properties of the asphalt.

MATERIALS

Asphalt.—The asphalt selected for the investigation was a steam-refined product prepared from heavy Mexican petroleum. Test results on the asphalt were as follows:

Specific gravity at 77°/77° F.	1.042
Flash point	570° F.
Softening point	134° F.
Penetration at 32° F., 200 g., 60 sec	16
Penetration at 77° F., 100 g., 5 sec	46
Penetration at 115° F., 50 g., 5 sec	198
Ductility at 77° F	100 +
Loss at 325° F., 50 g., 5 hours, per cent	0.03
Penetration of residue at 77° F., 100 g., 5 sec	40
Bitumen (soluble in carbon disulfide), per cent	99.77
Bitumen soluble in carbon tetrachloride, per cent	99.96

Mineral Aggregate.—A sheet asphalt mixture was selected for the investigation because of its greater convenience than a mixture containing coarse aggregate. The sand used in the mixture was a clean silica sand having the following sieve analysis:

	PER CENT	
Passing No. 200 sieve	4	
Passing No. 100 sieve	22	30
Passing No. 80 sieve	8	
Passing No. 50 sieve	14	34
Passing No. 40 sieve	20	
Passing No. 30 sieve	17	31
Passing No. 20 sieve	11	
Passing No. 10 sieve	3	
Retained on No. 10 sieve	1	

The mineral filler used in the mixture consisted of pulverized limestone having the following sieve analysis:

	PER CENT
Passing No. 200 sieve	82.2
Passing No. 80 sieve	17.6
Passing No. 50 sieve	0.2

Composition of the Sheet Asphalt Mixture.—The composition of the sheet asphalt mixture conformed to acceptable practice and was representative of mixtures used near Baton Rouge, La., where the investigation was conducted. The proportions of the ingredients of the mixture were as follows:

	PER CENT BY WEIGHT
Sand	76.5
Limestone dust	13.5
Asphalt	10.0
Total	100.0

PREPARATION OF MIXTURES

The mixing chamber in which the mixtures were made consisted of a cast-iron vessel (locally known as a No. 10 sugar kettle), hemispherical in shape, and having a top inside diameter of 17¼ in. and an inside depth of 10¾ in. This kettle was mounted on an asbestos-covered steel framework over a two-burner gas plate and was provided with an asbestos cover. The mineral aggregate was heated in this kettle to the desired temperature and the asphalt was heated separately. The mineral aggregate and asphalt were then thoroughly mixed together in the kettle by hand with the aid of a large spoon after which the mixture was removed from the kettle and allowed to cool.

All batches contained 20 lb. of material which provided sufficient asphalt, after extraction from each mixture, for several check tests.

The time required for mixing the batches at the various temperatures varied inversely with the temperature, but in each case the time taken was that period which was necessary to secure a thorough mixture. The actual time consumed in mixing each batch is shown in Table I.

EXTRACTION OF THE ASPHALT

The asphalt was extracted in accordance with the following methods:

Method I.—Each 20-lb. sample of sheet asphalt was divided into two equal portions and each portion was placed in a 4000-cc. beaker.

Carbon disulfide was then poured into each beaker in sufficient quantity (approximately two quarts) to cover the mixtures. The mixture and the solvent was stirred five or six times on the first day to thoroughly wash the mineral aggregate. The beaker was then allowed to stand for at least 48 hours, after which the solution was carefully decanted and filtered through a series of three 250-cc. funnels, using absorbent cotton as the filtering medium. This filtration removed practically all of the fine sediment in the asphalt. The material left in each beaker was subjected to a second washing conducted in the same manner. The solution was then placed in 2-qt. jars which were tightly covered and allowed to settle for 48 to 72 hours, after which the solution was finally filtered through an asbestos mat in a Gooch crucible to remove the last traces of sediment.

TABLE I.—RESULTS OF TESTS SHOWING THE EFFECT OF THE MIXING TEMPERATURE ON THE PHYSICAL PROPERTIES OF AN OIL ASPHALT IN A SHEET ASPHALT MIXTURE.

	Original Asphalt	Properties of Heated (Heated Only) and Recovered Asphalt, Various Mixing Temperatures											
		250° F.		300° F.		350° F.		400° F.		450° F.		500° F.	
		Heated[a]	Recovered[b]	Heated[a]	Recovered[b]	Heated[a]	Recovered[b]	Heated[a]	Recovered[b]	Heated[a]	Recovered[b]	Heated[a]	Recovered[b]
Temperature after mixing, deg. Fahr.	....	.....	260	.....	290	.....	343	.....	385	.....	443	.....	492
Time of mixing, minutes	....	.....	9	.....	8	.....	6½	.....	6½	.....	5	.....	2½
Penetration at 32° F	16	16	14	16	13	16	13	16	13	16	13	15	11
Penetration at 77° F	46	46	40	46	36	46	35	46	34	46	33	45	24
Penetration at 115° F	198	195	146	196	127	192	126	193	115	190	100	192	63
Softening point, Ring-and-Ball Method, deg. Fahr	134	134	142	134	145	134	144	134	144	134	147	135	162
Ductility at 77° F	100+	100+	100+	100+	100+	100+	107	100+	49	100+	36	100+	8

[a] Tests made on asphalt after heating to mixing temperature.

[b] Tests made on asphalt after recovery from the sheet-asphalt mixture.

Method II.—A 1000-g. capacity Rotarex centrifugal extractor was used in this method. Eight hundred grams of the sheet asphalt were used at one time and washed 6 to 8 times with 250 cc. of carbon disulfide. The solution obtained by these washings was placed in 2-qt. jars for 48 to 72 hours and then filtered through an asbestos mat in a Gooch crucible.

Both of these methods were used alternately. Method I is preferable to Method II; it requires less solvent and hence gives a heavier solution which is more easily distilled; the amount of sediment to be removed by the Gooch crucible is much less; and it requires a great deal less work than Method II.

Selection of the Solvent.—Carbon disulfide was used as the solvent for extracting the asphalt from the mixtures. The discussion of the previous paper referred to the possibility of using benzol in place of carbon disulfide. Careful consideration was given in this investigation to the use of other solvents and a large amount of work was done

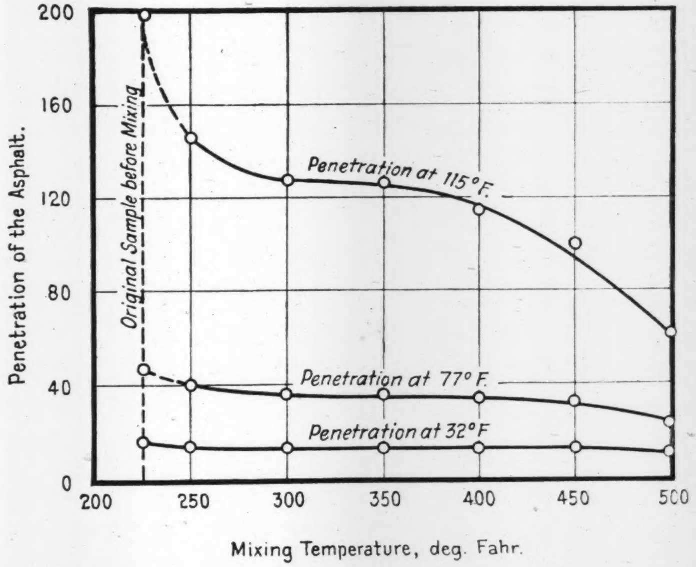

FIG. 1.—Effect of Mixing Temperature on the Penetration of an Oil Asphalt.

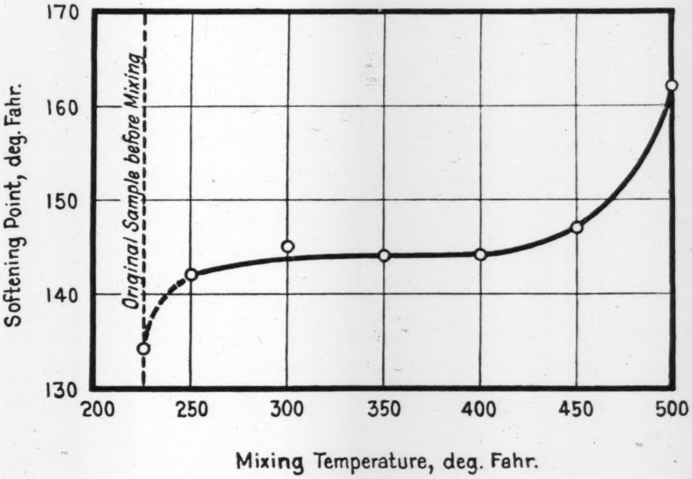

FIG. 2.—Effect of Mixing Temperature on the Softening Point of an Oil Asphalt.

with benzol. We found that we could not obtain consistent results with benzol extractions. A large number of solutions of benzol and the original asphalt were prepared and the asphalt recovered by the distillation method described herein. For some reason, which we are unable to explain, the benzol consistently could not be removed as well as the carbon bisulfide. We therefore abandoned its use. Carbon tetrachloride was not used because of its tendency to decompose under the action of heat.

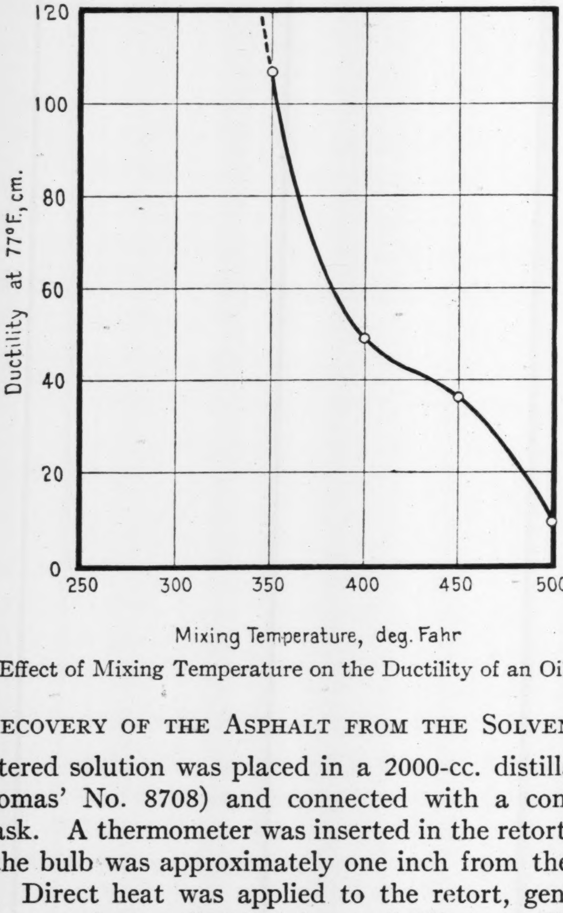

FIG. 3.—Effect of Mixing Temperature on the Ductility of an Oil Asphalt.

RECOVERY OF THE ASPHALT FROM THE SOLVENT

The filtered solution was placed in a 2000-cc. distillation retort (A. H. Thomas' No. 8708) and connected with a condenser and receiving flask. A thermometer was inserted in the retort so that the bottom of the bulb was approximately one inch from the bottom of the retort. Direct heat was applied to the retort, gently at first, and slowly increasing until the solvent distilled steadily, but not streaming, and until the temperature of 300° F. was attained, at which point foaming had practically ceased. The heat was temporarily removed at this point (on account of the fire hazard) and the discharge end of the condenser was connected with a filter pump (attached to a water outlet) and the distillation was continued with the aid of a partial vacuum, raising the temperature of the asphalt to 500° F. at the rate of 5 to 7° F. per minute. The heat was then removed and, without stopping the filter pump, the partial vacuum in the retort was released by slowly removing the thermometer from the retort. The filter pump was then stopped and the recovered asphalt poured from the retort into penetration cans and other testing devices. The precaution of releasing the partial vacuum in the retort by removing the thermometer is essential. If the filter pump is shut off before releasing the vacuum, water will be sucked into the retort.

DISCUSSION OF TEST RESULTS

The results of tests of the asphalt are given in Table I.

The effect of the mixing temperature on the penetration, softening point and ductility of the asphalt are shown graphically in Figs. 1, 2 and 3. The change in penetration at 32° F. was not marked but was significant at 77° F. and 115° F. The effect on the softening point was very significant. The ductility curve shown in Fig. 3 is not complete because the length of the ductility machine did not permit obtaining the ductility for values over 150 cm.

Figures 1, 2 and 3 show that the mixing operation hardens the asphalt, increases its softening point, and decreases its ductility at 77° F. The similarity of these changes in the physical properties of the asphalt to the changes produced by air-blowing of asphalt is very great and it is possible that the action which takes place is of a similar nature. In the mixing of asphalt and fine mineral aggregate a very large surface area aggregate, covered with a thin film of asphalt, is brought into contact with hot air. The air thus has an opportunity to accomplish results which are similar to those produced by direct blowing.

The effects shown in Table I and in Figs. 1, 2 and 3 cannot be attributed to the temperature to which the asphalt was heated prior to mixing it with the mineral aggregate. Tests were made to determine the effect of the heating temperature and the results of these tests are given in Table I. The effect of the heating temperature, even when as high as 500° F., was neglible.

The figures illustrating the effect of the mixing temperature on the softening point and ductility of the asphalt indicate that mixing above certain temperatures will cause changes which are detrimental. This critical temperature, in the case of softening point, is about 400° F. The ductility of the asphalt was greatly reduced when the mixing temperature exceeded 350° F. So far as could be determined in the laboratory no charring or burning took place within the range of temperature which was used, but the ductility, softening point and penetration tests do indicate that harmful effects may result when the temperature exceeds 350 to 400° F. in the case of the one asphalt which was studied. These results bear out rather commonly accepted opinion relative to the maximum mixing temperatures which should be used with Mexican petroleum asphalt.

It is undoubtedly true that all asphalts are not affected in the same way. This investigation dealt only with Mexican petroleum asphalt. The method of recovering asphalt from mixtures which has been developed provides a means whereby the asphalt in pavement mixtures may be examined to determine the effect of the mixing temperature.

CONCLUSIONS

This investigation covered only one type of asphalt and only one paving grade in that type and the conclusions which may be drawn from the test results may be applied to this one asphalt only. The results of the tests show that the penetration and ductility of the asphalt were decreased and the softening point was increased by the mixing operation and that these changes are affected by the mixing temperature. The method offers a satisfactory means of determining the effect of the mixing temperature on asphalt.

DISCUSSION

MR. T. H. ROGERS.[1]—I should like to ask Mr. Hubbard if he has obtained results that confirm the work of Mr. Bateman.

MR. PRÉVOST HUBBARD.[2]—I have not followed up the method which Mr. Bateman outlines, but in general I am in agreement with his conclusions, which seem to be quite in line with the generally accepted ideas of what happens when a mixture is made at too high a temperature.

There is just one point in connection with the conclusions reached by Bateman upon which I should like to comment, and that is his statement: "It is undoubtedly true that all asphalts are not affected in the same way." Personally, I doubt very much if that is so literally. I think that all asphalts are susceptible to oxidation and that when the asphalt is distributed in thin films over mineral aggregates, high temperatures cause much more rapid oxidation than is produced even in a regular oxidation process. So far as I know all asphalts are susceptible to oxidation, but the extent of oxidation which will occur under a given set of conditions may vary with the different asphalts. All asphalts are probably affected in the same way but there may be a difference in the extent to which different asphalts are affected when they are subjected to excessively high temperatures during mixing.

MR. ROGERS.—It is stated in the paper that the test portions of asphalt were maintained at the several temperatures a sufficient length of time to obtain a proper mixture. The batches consisted of 20 lb. of material, so it seems that an appreciable length of time was involved, and that it was somewhat in keeping with actual practice by which asphalts would be applied.

I am wondering whether it is not felt by the authors or others that a 5-hour evaporation test is a substantial measure of the changes such as are shown undergone in these tests.

MR. HUBBARD.—That does not seem to check because there was only 0.03 per cent loss at 325° F., and at that temperature there were no changes in characteristics, but at higher temperatures a decided change in characteristics was produced.

MR. ROGERS.—Apparently the results of the 5-hour evaporation test at 325° F. are more severe than this mixing at 325° F., but the point I was making is this: Is not the 5-hour evaporation loss at 325° F. a measure of what might be expected at some higher temperature of mixing, 450 or 500° F.? It results in the same kind of change, at any rate, as regards the effect on the properties. I am referring to the tests, such as penetration and ductility, on the residue (from the evaporation test) as significant and not the actual percentage of evaporation.

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[1] In Charge of Analytical Laboratory, Standard Oil Co. (Indiana), Whiting, Ind.

[2] Chemical Engineer, The Asphalt Assn., New York City.

MR. HUBBARD.—Of course, that will have to be determined by investigation along the same lines, using different types of asphalt. Only one Mexican asphalt was used, I believe, but the authors are trying to work out the same thing using different types of asphalt.

MR. A. W. DOW.[1]—I think this is a most timely paper. The damage done to asphalt pavements by laying them at too high a temperature has been really very great in the past few years. This is owing to the fact that some cities and states are laying stiffer mixtures than they have in the past, and so to overcome the difficulty in working on the street they have raised the temperature so as to make it more plastic.

I read a paper last winter before The Asphalt Association in which I emphasized this fact and showed that even though an asphalt is not actually burned at a temperature of 375 or 400° F. yet it is badly damaged by over hardening. I might answer Mr. Rogers relative to the comparison of the loss in the heating test with the test described in this paper. The loss on heating test is made by exposing 50 g. of the asphalt cement in a hot air oven at a temperature of 325° F. for 5 hours. In this test, the asphalt cement being in bulk, the evaporation and oxidation takes place on only a comparatively small surface. In the case of the overheated mixture we may have approximately 90 per cent of mineral matter with 10 per cent of asphalt cement. This mixture is tossed into the air in the mixer exposing each small grain of sand with its minute coating of asphalt cement to evaporation and oxidation. With this large surface exposed the hardening takes place very rapidly. It can readily be seen that the exposure in the hot mixture is far more severe than the exposure in the loss on heating test at 325° F.

MR. J. S. MILLER, JR.[2]—The authors are working along a line that is of great interest to us all. It is unfortunate, however, that they are using the character of apparatus that they specify, consisting of a cast-iron vessel (locally known as a No. 10 sugar kettle), and using a spoon for mixing. Undoubtedly some vital changes take place in asphalts in the mixing operation. It is a question in my mind whether the oxygen in the air is the only factor, but in order to have these results comparable with practice it would be desirable for the authors to work with a standard type of pug mill mixer, so as to get the full effect of the air, which I think is very much greater in practice than is shown in this character of kettle which has depth without great width; and in using a spoon I doubt if they are exposing the mixture to the air as much as in actual practice, because in actual practice we have found the same changes at temperatures lower than 400° F., which would indicate that at 350 or even 325° F. one may get serious modifications of the physical properties of the asphalt under those actual practical mixing conditions that are not shown in this paper.

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[1] Member of Firm, Dow & Smith, Paving and Chemical Engineers, New York City.

[2] Director, Technical Bureau, The Barber Asphalt Co., Maurer, N. J.

MR. J. H. BATEMAN[1] (*authors' closure by letter*).—The effects of the mixing operation on the physical properties of an asphalt are much more severe than those caused by heating of a mass of the asphalt itself; also, the effects are influenced by the *mixing temperature*. Mr. Dow has covered this consideration very completely in his discussion.

Mr. Miller's criticism of the use of hand mixing is not borne out by tests conducted on a plant-mixed sample of fine-graded asphaltic concrete and reported in a previous paper.[2] Tables I and IV of that paper give the results of tests on asphalt recovered from laboratory mixtures and commercial plant mixtures. The effect of the mixing operation in each case was practically the same. As a matter of fact, the authors investigated the practicability of using a small pug mixer in this investigation, but abandoned the idea when it was found that the cost was prohibitive. The conduct of the mixing experiments at different temperatures in a commercial plant was also impracticable at the time they were made, as no commercial plant was then available.

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[1] Director, Engineering Dept., Barrow-Agee Laboratories, Inc., Memphis, Tenn.

[2] J. H. Bateman and Charles Delp, "The Recovery and Examination of the Asphalt in Asphaltic Paving Mixtures," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 465 (1927).

TESTS OF ELASTIC CAULKING COMPOUNDS[1]

By H. H. Dutton[2]

Synopsis

This paper deals with the results obtained thus far in an investigation of the properties of elastic caulking compounds being carried on through the cooperation of the U. S. Bureau of Standards and the Indiana Limestone Co., Bedford, Ind., at the Bureau of Standards, Washington, D. C.

Samples of all the well-known materials in present-day use are under examination to determine their various properties such as resistance to extreme heating and cooling conditions, tendency toward staining stonework along joints, rapidity of hardening and drying out, elasticity after exposure, and durability after long-time exposure in actual construction joints. Over sixty samples from about thirty manufacturers have been included in the tests.

Results obtained thus far indicate that most of the materials are of poor quality and do not serve the purpose for which they are intended. Based upon the defects observed in these materials and the results obtained with those of apparently satisfactory composition, a discussion is given of the desirable features which should be present in good compounds and suggestions are made as to possible improvement in the manufacture of such products.

In considering the use of the so-called elastic caulking compounds in present-day building construction several factors must be included. The three outstanding qualities which a good material must have are adhesiveness, elasticity[3] and permanence. Also important but relatively less so are non-staining oil content, durable color harmonizing with the masonry and ease of application on the job. The different commercial products now on the market have presumably been designed to have the properties set forth above, but few of them have been observed to be entirely satisfactory in actual use. To provide data on this subject an investigation is now being carried on under the cooperative research program undertaken by the U. S. Bureau of

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[1] Publication approved by the Director of the Bureau of Standards of the U. S. Department of Commerce, Washington, D. C.

[2] Research Associate for the Indiana Limestone Co. at the U. S. Bureau of Standards, Washington, D. C.

[3] In the trade a caulking compound is said to have a satisfactory degree of elasticity when it will withstand, without cracking, the deformations to which it is ordinarily subjected in service. The deformation may be both elastic and plastic in nature, but according to this use of the term elasticity, no distinction is made.

Standards and the Indiana Limestone Co. of Bedford, Ind. A discussion of the factors involved in the use of caulking compounds and a summary of the results thus far obtained in this investigation follows.

In coping joints, horizontal joints in water tables and cornices, and in vertical joints provided in long walls for expansion and contraction movements, caulking materials of an elastic nature should be used. Much of the discoloration and efflorescence on stonework can be traced directly to the seepage of water through open joints in the locations just mentioned. If a material could be inserted in place of the usual pointing mortar in such joints which would be sufficiently adhesive to cling to the stone on each side of the joint, elastic enough to follow the movement of the stone without cracking, and also capable of maintaining these elastic and adhesive properties during years of exposure, the problem of water intrusion through joints would be well taken care of. The material must have sufficient bonding strength so as not to shrivel or crack away from the stone under the heat of the sun or the low temperatures of winter weather. If squeezed between the blocks when expansion occurs or if stretched when contraction takes place in the structure, the compound must be capable of returning with the stone to its original position without damage to itself.

Almost without exception the commercial caulking compounds consist of inert fillers mixed with some sort of oil to give the required plasticity. The oil is sometimes of such a nature that the stone may be stained by it when the material is put in a joint. Such oil stains not only are likely to present an unsightly appearance due to their own color, but if sufficient oil is absorbed along the joint edges, dust and dirt in the air may be accumulated by the oil-soaked stone to further disfigure the building. It is important, therefore, that the oil content be non-staining in character and not present in such form or quantity that the stone may absorb it readily.

When using a caulking material in joints which are visible from the ground, the color must be such as to either match the stone or be of only slightly different color tone. Where the joints to be filled are not visible, the colors are not important and even black compounds would be entirely suitable. Upon exposure to the weather the original colors should retain their tones without fading and here again the oil content must be considered, since the drying of the surface due to the oxidation of the oil may cause fading or, if the oil does not dry appreciably, the dirt collected on the material may cause an undesirable darkening effect.

After all the above conditions have been met by a particular compound, there yet remains another feature which may of itself decide the usefulness of the product. This is the question of facility of application on the job. The workmen must be able to handle the compound without difficulty, either in filling the joints or in finishing off the surface. An exceptionally sticky material may adhere so tenaciously to the caulking tools that the placement work becomes slow and tedious with a consequent high labor cost. Also, smearing of the edges of the joints is almost certain to occur on any job and if the material cannot be easily cleaned off the stone due to its excessively adhesive nature, the stonework will be disfigured by the irregular outlines of the uncleaned joints. Trouble is sometimes encountered in finishing joints when the material cannot be smoothed off easily and pressed firmly against the edge of the joints.

To produce a caulking compound which will successfully meet the conditions discussed above is difficult, yet there are many materials to be found on the market which are described by the makers as being "permanent," "unchangingly elastic," and as having all the necessary qualifications for a perfect compound. To determine, if possible, the actual merits of the commercial products when tested impartially, the investigation now being carried out was started by gathering samples from different parts of the country. With few exceptions all of the sixty or more samples were bought in the open market by a Washington cleaning and pointing contractor, who asked for small quantities of material which were to be suitable for use on Indiana limestone of both buff and gray color. The samples were turned over to the laboratory as received in the original packages and it is reasonably certain that the materials are representative of usual commercial products. Other samples have been and are being obtained from time to time and in all cases the endeavor has been to purchase the material in use for ordinary caulking purposes. Since the first tests were made several manufacturers have submitted special samples in an effort to show an improvement in their products.

The laboratory work included chemical analyses and physical tests, and has been supplemented by service tests. Analyses have been made to determine the content of the materials, and physical tests are used to find such properties as resistance to high and low temperature, elasticity, staining of the stone, and tendency to run or slump from vertical or inclined joints. For the service tests samples of each material are being used as actual caulking in the coping joints of stone in the Industrial Building at the Bureau of Standards.

The composition of most of the compounds is somewhat like ordinary glazing putty, the principal constituents being an oil or combination of oils together with a filler of whiting, lithopone, or other inert substance. Asbestos fiber is often used, either in a finely ground state or in rather coarse condition. The colors are obtained by the use of various pigments and a pleasant odor is sometimes produced by the use of pine oil or other aromatic oils. The oils are usually of a semi-drying nature and are difficult of exact identification by analysis, since mixtures of two or more oils may be used or the presence of pine oil or other essential oils may eliminate the characteristics which serve to identify the principal oil. Mineral oils are rarely used. In some compounds small amounts of rubber and rosin are also found.

Since the prime requisite of a caulking compound is permanence of elasticity, no oils should be used which will harden in weathering. When drying oils are found in a material it can easily be predicted that the compound will fail on exposure to the weather, and in the test work thus far this fact has been well proved. Since a comparatively short exposure will serve to show the tendency of a drying oil compound to harden and lose its elasticity, it is curious to note the claims of several manufacturers of such materials that their products are "permanently elastic."

Due to the fact that installations of caulking compounds are made by hand with caulking tools and also with so-called "guns," most of the compounds are supplied by the makers in two grades of different stiffness. For use in the guns more oil is required to enable the material to be easily forced from the guns, while for hand work a stiffer compound is necessary. While a higher oil content tends to make the material shrink more, it is a question whether that feature is not offset by the advantage gained by forcing the compound into the joints under some pressure.

Since the value of a material depends to a large extent upon the rate at which the oil hardens in weathering, a heating and cooling test was planned to give data on this point. Two duplicate sets of specimens were prepared by placing the different materials in $\frac{1}{2}$-in. slots $\frac{3}{4}$ in. deep cut in small stone blocks. One set of these blocks is being heated at 120° F. for 1200 hours and the other set is being cooled at 15° F. for the same length of time. Heating at 120° F. is thought to be equivalent to the average maximum sun temperatures in moderate climates and cooling to 15° F. is equivalent to exposure at winter temperatures. The specimens in these tests are being examined from time to time and photographs are being taken to record their condition. The 1200-hour limit has been reached on a number of sets of these materials and it is interesting to note that some of the compounds seem to lose their elastic properties equally rapidly whether in the heating or freezing test.

To compare the staining properties of the materials on both buff and gray Indiana limestone, slabs of each kind of stone 6 in. wide were slotted with $\frac{1}{2}$-in. slots $\frac{3}{4}$ in. deep, 5 in. apart along the length of the slabs. These slots were filled with the compounds and the slabs set on racks inclined at 60 deg. to the horizontal. These specimens not only provide staining tests but also give an opportunity to observe the tendency of the materials to flow or slump from joints and any changes in color which may occur on exposure. One set of slabs is set up in the laboratory while the other is placed outdoors on the roof of the laboratory building.

The coping stone joints of the parapet wall of the Industrial Building of the Bureau were originally pointed with the usual cement mortar. As the caulking compound samples have been secured the coping joints have been cut out and filled with the compounds by experienced men furnished by the local contractor who is obtaining the samples. The workmen put the materials in the joints as would be done on a commercial job and their comments on the ease or difficulty of handling the compounds furnish good data on the commercial suitability of the materials. Two joints are filled with each material, one being made $\frac{1}{2}$ in. wide by $\frac{3}{4}$ in. deep and the other $\frac{3}{8}$ in. of the same depth. Most of the joints have been finished for over twenty months and others are being prepared as the samples come in. The condition of the various materials after exposure in this manner shows very well what may be expected of similar compounds in practical construction.

After the examination of the first group of samples it was decided to omit the elasticity tests until after some of the other work had been done, since it seemed probable that some materials would fail in other tests and time would be lost in making the elasticity tests immediately. This forecast was well justified by the results of the coping stone joint, staining and slump tests, for only a short exposure was necessary to show the uselessness of more than 75 per cent of the first group of over 50 samples.

For the elasticity tests of those materials which warrant it, a set of two small stone blocks and a base block will be prepared to form a joint which can be compressed or extended as required. One small block is rigidly bolted to the base block and the second small block so arranged that it can be set at a distance from the rigid block to form a joint of proper width. The second block may then be moved towards or away from the rigid block by measurable amounts so that compression and tension may be exerted on the caulking materials. Evidence of the failure of the joint in case the material is unable to follow the stone when the blocks are separated will be shown by leakage of water which is introduced against the back of the joint from cavities cut in the joint stones. The movements of the joint will correspond to those probable in actual building construction where masonry is subject to temperature and other movements. This test is expected to show the relative elasticity of the various compounds and also their adhesive and cohesive properties. Sets of these blocks will be prepared in duplicate, one set being kept in the laboratory and the other exposed to the weather. These blocks are now being put together, using only those materials which have shown some value in the other tests.

SUMMARY

Considering the data obtained thus far the following comments may be made:

1. Materials of the "putty" type exhibit poor adhesive qualities and tend to harden too rapidly to retain their elasticity for a reasonable time.
2. Materials of low melting point and of too liquid consistency sag and run from joints which are vertical or inclined at sharp angles. High oil content also causes early cracking and opening of the joints.
3. Stains may be caused by the presence of oils which are readily absorbed by the stone, although exposure to sunlight will in some cases render the stains unnoticeable.
4. Materials containing a higher percentage of oil and coarse asbestos fiber in proportion to the filler than the putty type, seem to have an advantage in that they tend to form a skin on the outside of the joint on exposure to the weather, this skin helping to protect the remaining compound from drying out.
5. Many of the compounds are difficult to place in joints because of their excessive adhesiveness, this feature increasing the labor costs. Smeared and unsightly joints are also likely to occur with this type.
6. The colors of many materials did not harmonize with the stone and changed greatly on exposure. However, for joints not ordinarily visible some compounds can well be used regardless of their color.
7. Considering the results of tests to date on over sixty samples from thirty different manufacturers, it appears that only four compounds show promise of meeting the requirements of an elastic caulking compound as set forth above, and of these four only three seem suitable for general use in all locations.

DISCUSSION

Mr. Cloyd M. Chapman.[1]—I should like to ask Mr. Dutton whether among these sixty samples there were any of the emulsified bituminous types, such as emulsified asphalt.

Mr. H. H. Dutton.[2]—There were two compounds which were of that nature. One of them lasted for approximately six months without showing any serious defect, while the other in three months, as I remember it, shrunk in on itself, so that the whole strip could be picked from the joint.

Mr. D. K. Boyd.[3]—It is important that architects and manufacturers of window and door frames recognize that caulking is not only an advantage but a necessity, so that the architect in detailing his window frames and his door openings will provide the reveal mold or the staff bead, as it is sometimes called, with a set-back from the masonry opening sufficient to allow the proper incorporation of a caulking compound. A great trouble which exists in many buildings to-day is that the architects and the manufacturers of either wood or metal frames feel that they can make a sufficiently wind-tight joint, by the contact of the wooden mold or the metal mold and the masonry opening. Now that is a physical impossibility. Even were it so the shrinkage of mortar would open up a crack which would permit the entrance of air, and such a small crack is impossible of successful caulking. Therefore, the originators of the design and of the frames should provide a space for caulking in the beginning.

Mr. H. S. Brightly.[4]—In line with the remarks of Mr. Boyd, there is another rather important point that has not been given enough consideration by architects in using caulking compounds, and that is, insisting that the joint is sufficiently wide and raked out to a depth sufficient to get in a body of material that can be depended upon as a water-tight seal. It is unfair to the caulking material and absolutely useless, no matter how good the material may be, to put in a smear of it only about $\frac{1}{4}$ in. in depth on top of a mortar joint. It might just as well be left out as put in in that way. The joint must be made of sufficient width, never less than a full $\frac{1}{4}$ in., and be raked out to a sufficient depth, usually not less than $\frac{3}{4}$ in., in order to give the material a suitable area of contact with the masonry and enough body to be effective.

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[1] Consulting Engineer, New York City.

[2] Research Associate of the Indiana Limestone Co. at the U. S. Bureau of Standards, Washington, D. C.

[3] Architect and Structural Standardist, Structural Service Bureau, Philadelphia, Pa.

[4] Director, Architects' Service Bureau, Indiana Limestone Co., Bedford, Ind.

Mr. Dutton.—In reference to Mr. Boyd's comments, the installation of caulking compounds is taken care of in many designs by cutting off the back of the staff bead on an angle so as to allow a sufficient space for a fillet of material ½ to ⅝ in. wide, to be laid in behind the staff bead when the building is erected. The caulking material is placed in this space underneath the bead and the bead replaced. That usually provides an excellent seal, and even if the frame does shrink away from the joint later on, there is sufficient caulking material in the joint to be of value after such shrinkage has occurred.

Mr. Boyd.—That is a good method but involves the window frame being sent to the building with the staff bead temporarily attached. It is a cause of great complaint not only of architects but mill men and painting contractors that after the frame is all set the staff beads have to be taken off, the caulking then applied, the staff bead returned to its place by someone other than the man who made the mill work, then the painter has to come along and do an extra job of puttying up nail holes and painting. This is all obviated when the staff bead is permanently attached to the window frame and is set away, in order to get the proper amount of caulking between the masonry and the mold.

I think attention should be called to the necessity of cleaning out the space where the caulking is to go, so as to free it of all the mortar, dust, sawdust and other materials, or rust, in the case of metal, to allow for perfect adhesion of the caulking compound. In other words, emphasis should be placed upon cleaning out the joint before the caulking compound is applied.

Mr. Dutton (*author's closure by letter*).—Architects and builders in general are realizing the importance of caulking to a greater extent than ever before and specifications for caulking are being written for the majority of the substantially constructed buildings. This investigation was conducted with the view to determining the merits of some of the caulking compounds now on the market and it would appear from the results obtained thus far that there are but few upon which the architect can rely. Testing of caulking compounds under specifications would be long drawn out and rather difficult to perform, but in view of the increasing use of these materials it would seem that the manufacturers themselves would do well to investigate the condition of their materials after some years of exposure and take such steps as may be necessary to make them more truly permanent and elastic.

THE USE OF A GROUP OF PERFORMANCE TESTS FOR EVALUATING RUBBER COMPOUNDS THAT MUST WITHSTAND REPEATED COMPRESSION

By Harlan A. Depew[1] and E. G. Snyder[1]

Synopsis

Test methods for the evaluation of solid tire compounds are described in detail. They consist in preparing two test cylinders of rubber 5 in. in diameter and $3\frac{3}{4}$ in. high for each cure, and vulcanizing over a range of cures. One cylinder is tested by pounding with a force of 2500 lb. three times per second and measuring the temperature rise. Test pieces are taken from the other cylinder and tested for abrasion resistance, tear resistance, and hardness. Accelerated aging tests are also conducted to determine the state and uniformity of cure.

Data is given comparing two solid tire compounds of different sulfur contents. The conclusions, although of interest, are secondary to the evidence that rubber testing has developed to a point where the laboratory can go a long way toward evaluating compounds for service.

Considerable progress has been made during the past few years in the development of laboratory testing methods, and in the interpretation and application of existing test methods. At present it is possible to go a long way toward evaluating compounds for various conditions of service and this paper is presented to show what can be done through the use of a group of performance tests; one of which is a comparatively new test, in evaluating compounds that must withstand repeated compression.

The tests used consist of:

1. An aging test, chosen primarily to determine the state of cure and the uniformity of cure.
2. An abrasion test, in connection with a tear and a hardness test in order to understand it more fully.
3. A pounding test in which a large block of rubber is repeatedly pounded and the temperature rise noted.

In so far as the results are concerned, these tests are of minor interest individually but of considerable value collectively. The first two are well known and will not be discussed in detail. However, the pounding test will be described since it is comparatively unknown.

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[1] Research Chemist, New Jersey Zinc Co., Palmerton, Pa.

The particular pounding machine described in this paper (see Figs. 1 and 2) is similar to, but somewhat more rugged than one used by the U. S. Rubber Co. A somewhat different device for repeatedly compressing smaller blocks has been described by Abbott.[1]

PROCEDURE

A half dozen uncured rubber blocks, a little less than 5 in. in diameter and a little more than $3\frac{1}{2}$ in. high, are prepared. Three of these blocks are made by rolling up a $3\frac{1}{2}$-in. strip of rubber at the calender until the proper diameter is reached. The other three are made up by a more tedious procedure. Disks of rubber $\frac{1}{10}$ and $\frac{1}{4}$ in. thick and 5 in. in diameter are cut from calendered sheet. These disks are talced and then built into a cylinder of the required size. For a mold, two $\frac{1}{4}$-in. plates are used with pieces of pipe 5 in. in inside diameter and $3\frac{3}{4}$ in. high. Two of the uncured rubber cylinders, one made by rolling and one by cutting out disks from the calendered sheet, are placed in the molds and a disk $\frac{1}{4}$ in. thick of hard rubber compound is placed on the bottom. Loose wadding is packed around the cylinders inside a wood frame so that air currents cannot affect them during

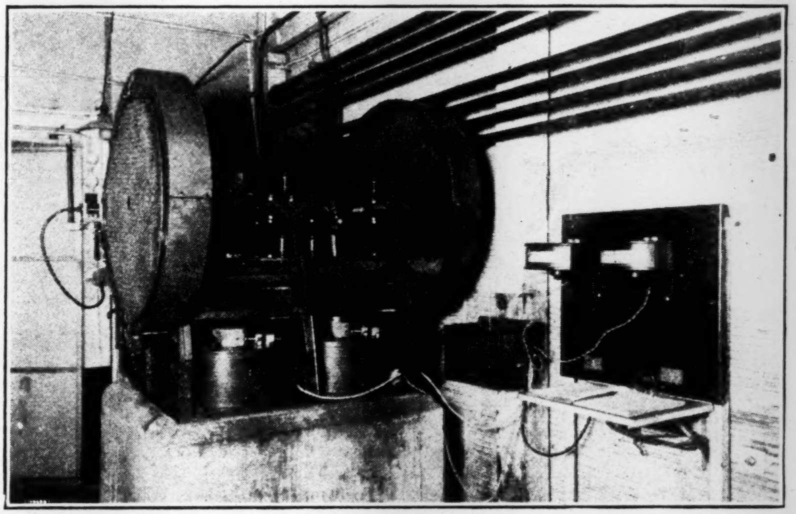

FIG. 1.—Pounding Machine with Test Cylinders in Place and Indicating Pyrometers Shown at the Left with Connecting Wires Leading to Thermocouples Embedded in the Centers of the Rubber Test Cylinders.

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[1] F. D. Abbott, "Machine for Testing Rubber Products Used to Absorb Vibration," *Industrial and Engineering Chemistry*, Vol. 20, No. 8, August 1, 1928, p. 853.

cure. They are placed in a cold press and given a cure with a step rise as follows:

5 minutes to 104° C.
15 minutes at 104° C.
2 minutes to 113° C.
18 minutes at 113° C.
20 minutes to 141.5° C.
total temperature rise of one hour
90 to 180 minutes at 141.5° C.

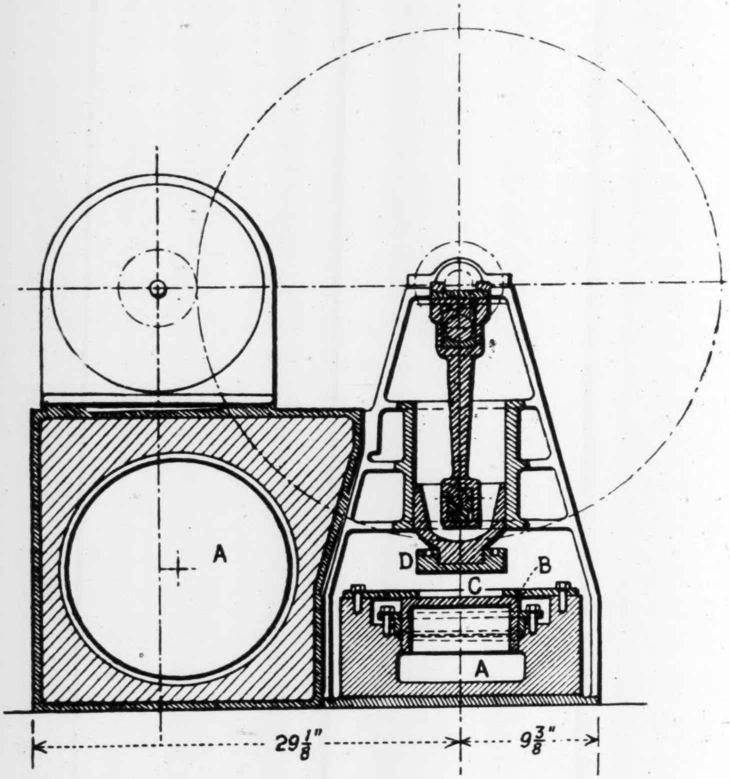

FIG. 2.—Showing Essentials of the Pounding Machine.

If the air pressure below the piston is 50 lb. per sq. in., the force holding the block in place is 2500 lb., and the hammer cannot strike a blow in excess of this figure since the piston will depress.

A, reservoir and air chamber carrying 50 lb. of air.
B, piston with 50 sq. in. of area.
C, location of the test cylinder.
D, hammer that pounds the test cylinder.

The rate of cure of the compound is preferably chosen so that the time for the technical cure in addition to the one-hour rise will be in the neighborhood of 120 to 150 minutes.

After curing, the block made by plying up disks is pulled apart along the talced surfaces, giving test pieces 5 in. in diameter and $\frac{1}{10}$ and $\frac{1}{4}$ in. thick, respectively. The $\frac{1}{10}$-in. thick slabs are used for tear and tension tests, both before and after accelerated aging; some slabs being tested at once and others after aging 14 days at 70° C. The $\frac{1}{4}$-in. slabs are used for abrasion tests, and, if a hardness test is made by a ball penetration method, these slabs are used for this test also.

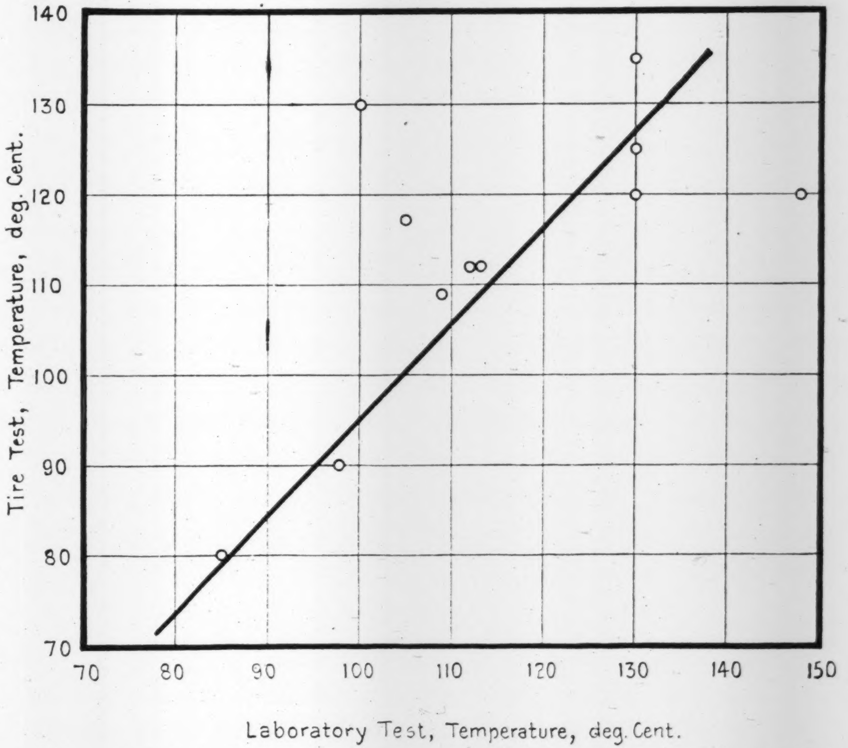

FIG. 3.—Comparison of the Temperatures Attained with Tires on the Test Wheel to Those Obtained on the Pounding Machine with Sections Cut from Solid Tires.

Laboratory Test.—100 minutes, 2500 lb., 3 blows per second.

Tire Test.—180 minutes, 4800 lb., 10 miles per hour.

The Bierer-Davis oxygen bomb test is probably preferable to the Geer oven test for certain types of compounds, but for high-grade compounds, such as are used in this case, the results by the two tests have been in close agreement and the more easily operated oven test has been adopted. (It has been found that thick rubber objects age more uniformly in the bomb test but these slabs are sufficiently thin so that this consideration does not apply here.)

The other block, made by rolling directly from the calender, is tested on the pounding machine and the temperature rise noted by inserting thermocouples by the Ashman awl method.[1] In this method the end of the couple is threaded into the eye of an awl needle and the couple inserted into place and left behind as the awl is withdrawn.

It is essential in the use of a test machine such as the pounding machine that there should be at least a rough correlation between the results obtained by it and the results obtained in the corresponding service. The data given in Fig. 3 show that such a correlation is obtained by comparing tests on cylinders cut from tires (Fig. 4) with wheel tests on duplicate tires. Two points are clearly off the curve but the results are quite satisfactory when it is considered that the tire from which the cylinder was cut may not actually be an exact duplicate of the tire used for the tire test, in spite of the fact that the two tires are from the same lot, and that the difference in windage cooling that occurs in tire testing, due to the different non-skid designs, is not effective in the case of blocks tested on the pounding machine. The appearance of a test block after failure (Fig. 5) is slightly different from that of a blown-out tire (Fig. 6) but in essentials, the failures are identical.

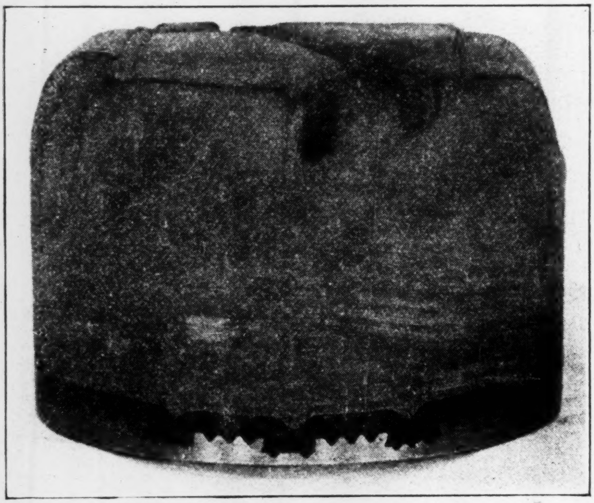

FIG. 4.—Test Cylinder Cut from a Solid Tire.

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[1] A. O. Ashman, "The Determination of Tire Temperatures," *Bulletin*, The New Jersey Zinc Co.

Solid tires are cured in a mold surrounded by steam, whereas these test cylinders which are cured in a press are only heated from the ends and there is the consideration of possible heat loss from the sides of the cylinder. This is small, however, as shown by the fact that the air space around the molds was found to be at approximately the same temperature as the steam in the press. The uniformity of cure that is obtained is shown in Table I. The tensile properties at

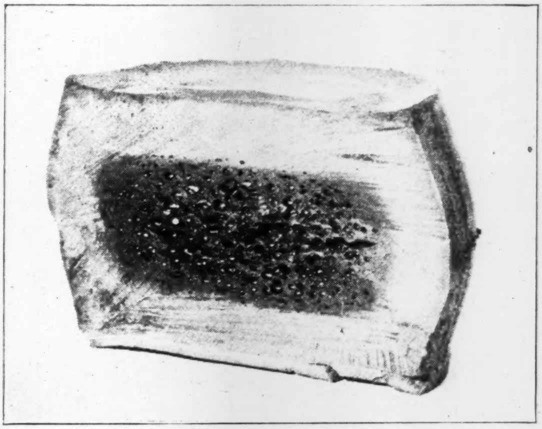

FIG. 5.—Example of Failure that Occurs when a Test Cylinder Receives More Severe Treatment than it is Built to Withstand.

Note the absence of sulfur bloom at the center where the block has reached a relatively high temperature.

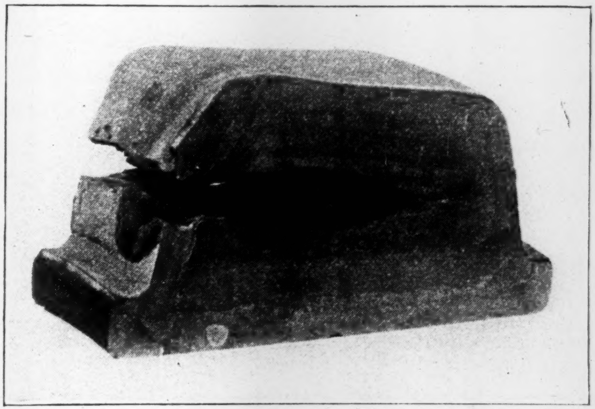

FIG. 6.—Example of a Blown-Out Solid Tire.

The heat causes the center to melt and the liquid core blows out. The failure is essentially the same as in Fig. 5.

various depths through the test cylinders are uniform, both before and after accelerated aging. The uniformity is as good as that obtained with solid tires.

SELECTION OF COMPARABLE CURES

In comparing compounds, it is necessary to make the comparison at comparable cures, and this involves the choice of a criterion of cure. The tensile optimum and the optimum area under the stress-strain curve are commonly used for this purpose. Another method is the amount of deterioration during accelerated aging. The decrease in tensile strength, percentage or absolute, is not entirely satisfactory since the tensile strength sometimes increases but the elongation, on the other hand, practically always decreases. After weighing the possibilities, elongation at break after aging was chosen and it was arbitrarily decided that the comparable cure would be that cure

TABLE I.—UNIFORMITY OF CURE IN A TEST CYLINDER AS SHOWN BY TENSION TESTS BEFORE AND AFTER AGING ON SLABS TAKEN AT DIFFERENT DEPTHS.

COMPOUND USED:	PARTS BY WEIGHT	PARTS BY VOLUME
Pale Crepe	460	100
Smoked Sheet	460	
Sulfur	37	...
D. O. T. G.	9.5	...
Stearic Acid	2.3	...
Medium Pine Tar Oil	4.6	...
XX Red Zinc Oxide	1680	30
Carbon Black	87	5

Time of Cure at 141.5° C., minutes[a]	Depth into the Rubber Cylinders from Which Test Pieces are Removed, in.	Before Aging		After Aging 14 Days at 70° C.	
		Tensile Strength, lb. per sq. in.	Elongation, per cent	Tensile Strength, lb. per sq. in.	Elongation, per cent
120	0 to 0.5	3150	500	1485	350
	0.5 to 1.5	3225	525	1510	405
	1.5 to 2.0	3100	450	1585	410
	2.0 to 3.0	3000	500	1480	390
	3.0 to 3.5	2780	505	1405	325
150	0 to 0.5	2700	475	1125	240
	0.5 to 1.5	2910	515	1310	290
	1.5 to 2.0	2700	490	1300	275
	2.0 to 3.0	2650	480	1250	265
	3.0 to 3.5	2395	440	1090	180
180	0 to 0.5	2120	400	1000	150
	0.5 to 1.5	2205	430	1060	160
	1.5 to 2.0	2310	435	1075	155
	2.0 to 3.0	2145	410	1065	150
	3.0 to 3.5	2035	415	960	130
210	0 to 0.5	1890	440	865	145
	0.5 to 1.5	2005	430	985	150
	1.5 to 2.0	1810	430	985	150
	2.0 to 3.0	1910	440	1015	145
	3.0 to 3.5	1755	420	875	130

[a] The time given is in addition to the one-hour temperature rise.

TABLE II.—CHECK TESTS ON THE SAME COMPOUND MADE AT INTERVALS OF ABOUT A MONTH.

COMPOUND USED:	PARTS BY WEIGHT	PARTS BY VOLUME
Pale Crepe	460	100
Smoked Sheet	460	
Sulfur	37	...
D. O. T. G	9.5	...
Stearic Acid	2.3	...
Medium Pine Tar Oil	4.6	...
XX Red Zinc Oxide	1680	30
Carbon Black	87	5

Date of Test	Time to Comparable Cure, minutes[a]	Tensile Strength		Elongation, per cent	Loads at				Relative Abrasion Resistance	Temperature of Block, deg. Cent.[b]	Compression of a Test Block Under 2500 lb. Load		After 14 Days Aging in Geer Oven at 70° C.			
					300 per cent Elongation		450 per cent Elongation						Tensile Strength		Elongation, per cent	Abrasion Resistance Units
		lb. per sq. in.	kg. per sq. cm.		lb. per sq. in.	kg. per sq. cm.	lb. per sq. in.	kg. per sq. cm.			in.	cm.	lb. per sq. in.	kg. per sq. cm.		
Nov. 5, '28	138	2540	178	490	1345	94	2240	157	143	83	0.46	1.17	1245	87	300	103
Dec. 12, '28	138	2840	200	485	1455	102	2475	174	147	86	0.48	1.22	1330	94	300	114
Jan. 16, '29	144	2575	181	470	1585	111	2500	176	158	86	0.45	1.14	1365	95	300	117

[a] The comparable cure was the one that had an elongation of 300 per cent at break after 14 days at 70° C. This is a very arbitrary choice, but it works well for this class of compounds. The time given is in addition to the time of temperature rise.

[b] After pounding three times per second with a load of 2500 lb. for two hours.

TABLE III.—COMPARISON OF FOUR AND SEVEN-PER-CENT SULFUR COMPOUNDS AT THE COMPARABLE CURE.

COMPOUNDS, PARTS BY WEIGHT:	4 PER CENT COMPOUND	7 PER CENT COMPOUND
Pale Crepe	460	460
Smoked Sheet	460	460
Sulfur	37	64
D. O. T. G	9.5	6
Stearic Acid	2.3	2.3
Medium Pine Tar Oil	4.6	4.6
XX Red Zinc Oxide	840	840
Red Label Kadox	840	840
Carbon Black	87	87

Amount of Sulfur in Compound, per cent	Time of Comparable Cure, minutes	Tensile Strength		Elongation, per cent	Load at 300 per cent Elongation		Goodrich Tear Test, lb. per Inch of Thickness	Abrasion Resistance	Temperature of Blocks, deg. Cent.	Compression of Blocks		After 14 Days Aging in Geer Oven at 70° C.			
												Tensile Strength		Elongation, per cent	Abrasion Resistance
		lb. per sq. in.	kg. per sq. cm.		lb. per sq. in.	kg. per sq. cm.				in.	cm.	lb. per sq. in.	kg. per sq. cm.		
4	139	2910	204	505	1300	92	338	163	79	0.35	0.89	1445	102	300	139
7	134	3040	222	520	1365	96	360	165	75	0.38	0.97	1590	112	300	147

which showed an elongation at break of 300 per cent after fourteen days in the oven at 70° C. This elongation is the average value obtained on test pieces cut from commercial solid tires. An absolute figure for elongation at break after aging rather than a percentage decrease is justified by the fact that the type of compounds used for this service will have approximately the same initial elongation.

TABLE IV.—SHOWING INFLUENCE OF CURE ON AGING.

	Time of Cure at 141.5° C., minutes[a]	Before Aging			After Aging	
		Tensile Strength, lb. per sq. in.	Elongation, per cent	Tear Resistance	Tensile Strength, lb. per sq. in.	Elongation, per cent
4-per-cent Sulfur Compound.....	120	3145	535	383	1785	415
	150	2765	490	313	1240	230
	180	2575	480	281	1135	195
	210	2235	285	228	925	180
7-per-cent Sulfur Compound.....	120	3195	545	404	1855	390
	150	2865	495	309	1295	200
	180	1760	340	195	940	110
	210	1325	300	119	810	100

[a] The time given is in addition to the temperature rise.

TABLE V.—RATE OF HEATING OF BLOCKS ON THE POUNDING MACHINE OVER A RANGE OF CURES.

	Time of Cure at 141.5° C., minutes	Time of Pounding, minutes							Compression of Blocks under 2500 lb. Load	
		0	30	60	90	100	110	120		
		Temperature, deg. Cent.							in.	cm.
4-per-cent Sulfur Compound...............	120	20	50	62	75	78	81	82	0.39	0.97
	150	21	48	63	71	74	76	78	0.33	0.84
	180	21	50	63	78	81	83	85	0.35	0.89
	210	23	62	88	106	109	112	113	0.41	1.04
7-per-cent Sulfur Compound...............	120	20	53	67	75	76	78	79	0.41	1.04
	150	22	48	59	67	69	70	71	0.34	0.87
	180	22	50	66	72	72	73	74	0.26	0.66
	210	20	50	75	86	89	92	94	0.27	0.69

NOTE.—A high-sulfur compound hardens continually with progressive vulcanization whereas a low-sulfur compound, which cures faster in the early part of the cure, hardens at first and then softens due to the degrading influence of heat on the vulcanized rubber as it is formed. The point at which softening exceeds the hardening is spoken of as reversion, but this is a meaningless expression because the softening has been taking place from the beginning of the cure, the rate depending on the extent of the vulcanization and the temperature. Even a high sulfur compound which shows continual hardening with cure is less hard than it would have been if no degradation had occurred.

Using this criterion of cure, Table II gives data showing the reproducibility of three tests. These three mixes covered a period of two months and were good checks.

In studying solid tire compounds, a lot of four compounds, three unknowns and a standard, are usually investigated. The investigations have included a comparison of different rubbers, the influence of small amounts of softeners, a comparison of different zinc oxides and other pigments, and the influence of high and low sulfur contents, with results that have been in line with what experience would predict.

AN EXAMPLE OF THE INTERPRETATION OF THE RESULTS OF THE TESTS—COLLECTIVELY CONSIDERED

For the purpose of illustration, data on the influence of sulfur content using the three tests earlier referred to, are given in Table III. It is evident that at the comparable cures chosen, there is little choice between 4 and 7 per cent of sulfur although there was an edge in favor of the 7-per-cent sulfur compound.

One of the arguments for low sulfur contents is that a more uniform cure is obtained with thick objects, and that errors in the time and temperature of the cure will be less serious. It was found that with the long slow cure used, the uniformity of cure was as good with 7 per cent sulfur as with 4 per cent sulfur, but Table IV shows that the change in tensile properties over a range of cures was greater for the 7-per-cent sulfur compound.

Accordingly, from the viewpoint of uniformity of cure and state of cure, there is no choice as to how much sulfur is used, but if the steam temperature around the mold is not properly maintained, the 7-per-cent sulfur compound will be affected more than the 4-per-cent compound.

In order to make the data on the subject of cure more complete, the temperatures attained by blocks of various cures under pounding is given in Table V. These data show that the 7-per-cent sulfur compound consistently produced lower temperatures over a range of cures. The lower temperature for the high-sulfur compound was reasonable because the vulcanized rubber in this compound would be degraded less by the heat. The larger amount of soft degraded rubber with the low-sulfur compound allowed the test cylinder to compress more, thereby doing much work, and the poorer resiliency further increased the heat generation.

Comparatively little has been said about the tear tests because the tear resistance was about the same for the two sulfur contents, but this is a serious consideration in most comparisons. Cracks developing through the body of solid tires with great chunks of rubber tearing off is a major factor not to be neglected, as is shown by the casual observation of many tires in service.

From a consideration of the various factors involved in the above example of the application of the various tests to a study of the effect of sulfur content, it might be concluded that:

1. Low sulfur contents tend to minimize variability in cure due to materials or vulcanizing temperatures;

2. The sulfur content does not affect the uniformity of cure throughout the tire when sufficiently long cures are used; and

3. High sulfur contents tend to lower the temperature of the tire in service.

The conclusions drawn from the data presented are secondary, however, to the thought that such rubber testing methods have developed to a higher stage of usefulness than is generally admitted, and that it is possible to go a long way toward evaluating compounds for different conditions of service by the use of existing performance tests, with the possible addition of some other direct simple test that applies specifically to the product under development.

A NEW TEST FOR MEASURING THE FIRE RESISTANCE OF WOOD

By T. R. Truax[1] and C. A. Harrison[2]

Synopsis

The object of this paper is to describe the Forest Products Laboratory fire-tube apparatus and to present a tentative procedure for testing the inflammability of wood and the effectiveness of various fire-retardant treatments. The fire resistance is expressed in percentages of loss in weight and increase in temperature resulting from the combustion of a specimen of uniform dimensions. The method gives definite numerical values which can be duplicated. Results are presented to show the development of the apparatus and procedure, typical variations in fire resistance between treated and untreated wood, and comparisons with other methods of test. As a measure of the property of wood to support combustion, that is, to cause the spread of flame and contribute to an increase in temperature under fire conditions, the Forest Products Laboratory fire tube test is reliable, consistent, and quickly and easily made.

The Forest Products Laboratory has recognized for a number of years the necessity of developing and standardizing test methods for fire resistance of wood. In 1927, Dunlap of the Forest Products Laboratory and Cartwright[3] of the National Lumber Manufacturers' Association discussed briefly the existing test methods and the forms of tests which they considered essential in evaluating the fire resistance of combustible building materials. Since that time, with the cooperation of the National Lumber Manufacturers' Association, the Forest Products Laboratory has been able to continue studies in the development of apparatus for the purpose.

Considered as to fire-resistance, the important properties of wood, and for that matter other building materials, are (1) the extent to which it resists combustion, (2) its capacity to retain strength properties under fire conditions, and (3) its resistance to the passage of heat through it. The relative importance of these properties varies for different uses. The first property is of primary importance from the general standpoint of reduction of damage by fire; the second is significant in timber for structural purposes; and the third is of importance for doors and partitions in buildings.

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[1] Senior Wood Technologist, Forest Products Laboratory, Madison, Wis.

[2] Assistant Engineer, Forest Products Laboratory, Madison, Wis.

[3] M. E. Dunlap and F. P. Cartwright, "Standard Fire Tests for Combustible Building Materials," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 534 (1927).

Obviously, small scale laboratory tests on samples of treated wood are not a sufficient measure of the fire resistance of completed structural assemblies, such as doors, window frames, sash and floors. Performance ratings for such assemblies must be arrived at by full-scale tests under the customary or the standard methods in use for the purpose. Laboratory tests do, however, afford a practical means of evaluating the fire resistance of wood entering into built-up assemblies. Such tests must not only give information on one or more of the important properties of fire-resistant wood, but they should be of such character that they can be quickly and easily made, will give results that can be expressed numerically, and are capable of duplication.

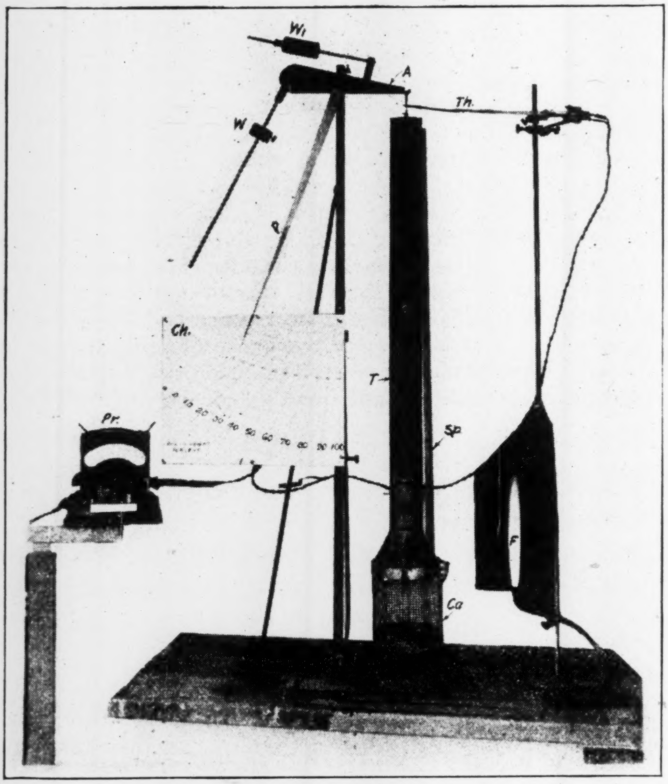

FIG. 1.—The Forest Products Laboratory Fire-Tube Apparatus.

The Forest Products Laboratory fire-tube apparatus, described in this paper, was developed in an attempt to find a satisfactory numerical measure of the resistance of wood to combustion and particularly of its resistance to the spread of flame and to a rise in temperature. It was desired that this method of test should give results that could be duplicated with a reasonable degree of certainty and that it should measure relatively small differences in degree of fire resistance. The fire-tube test is not proposed as a measure of all the properties of fire resistance. Other tests will be necessary to evaluate properties, such as the retention of strength under fire conditions, flame penetration, and heat insulating value. The effect of fire retardants on the finishing, hygroscopicity and gluing of wood, and on corrosion of metals is also important to the user of fire-resistant wood that require special testing methods.

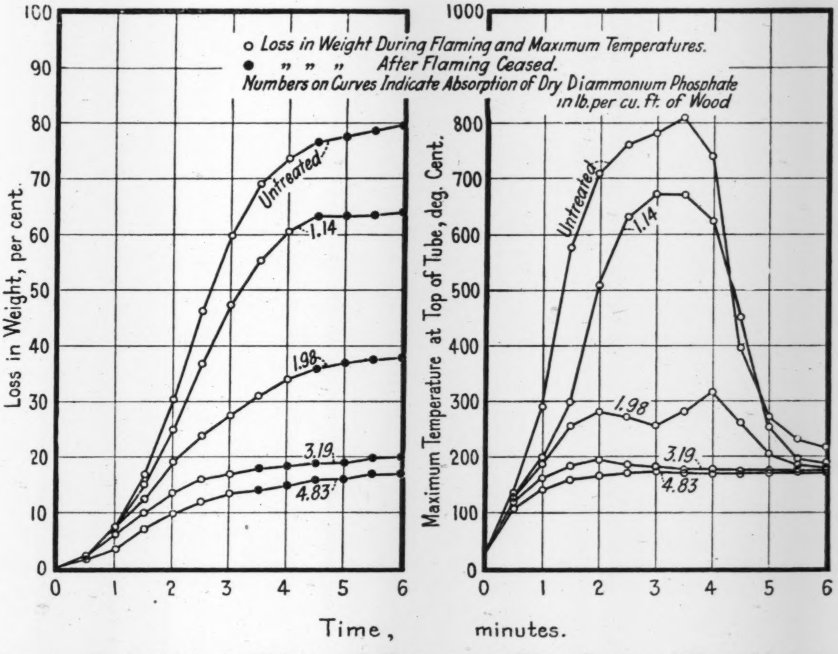

FIG. 2.—Typical Loss-in-Weight and Temperature-Time Curves of Tests Made in the Forest Products Laboratory Fire-Tube Apparatus.

FIRE-TUBE APPARATUS

The apparatus[1] (see Fig. 1) consists essentially of an open galvanized iron tube, *T*, in which the specimen, *Sp*, suspended from a balance arm, *A*, is burned. To the balance arm is attached a pointer, *P*, which indicates on a chart, *Ch*, the percentage of loss in weight as the burning proceeds. The apparatus is also equipped with a thermocouple, *Th*, and pyrometer, *Pr*, for measuring the temperature at the top of the tube. The specimen shown at the side of the tube is hung in the center of the tube during test. A gas flame, *F*, is applied to the lower one-fourth of the specimen. A wire screen cage, *Ca*, attached to the bell-shaped bottom of the tube, serves to catch burning embers that fall during the test. A double row of slots on opposite sides of the tube allows the entrance of air and permits the operator to watch the progress of the burning.

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[1] Original design by M. E. Dunlap of the Forest Products Laboratory.

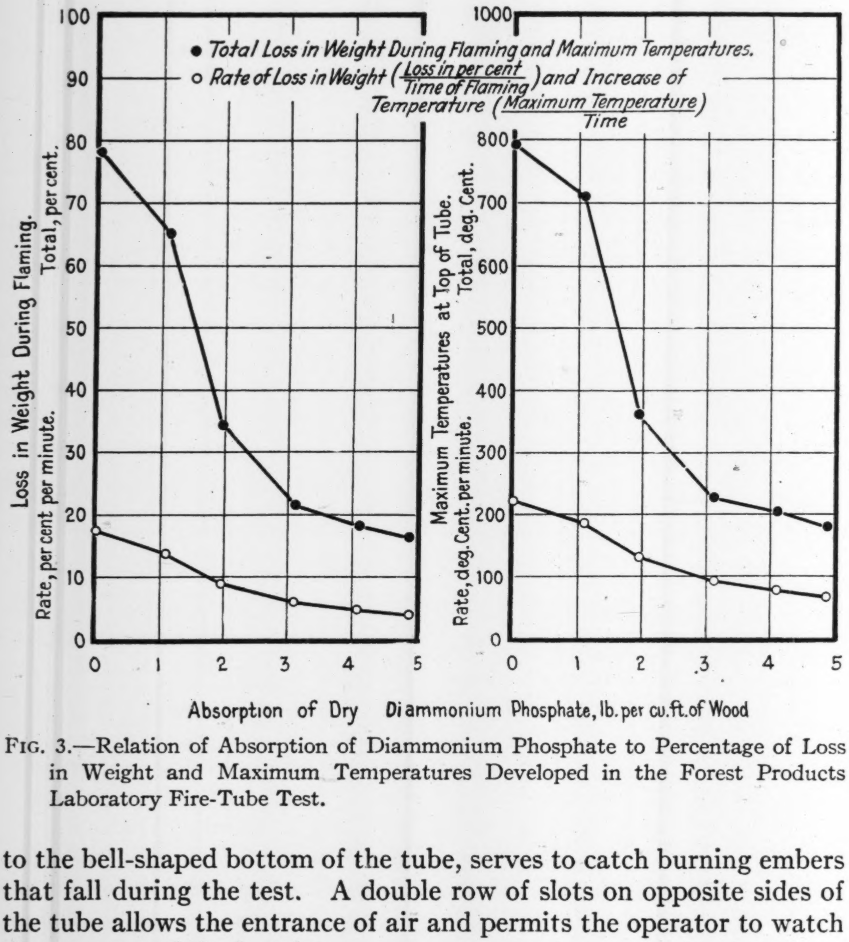

FIG. 3.—Relation of Absorption of Diammonium Phosphate to Percentage of Loss in Weight and Maximum Temperatures Developed in the Forest Products Laboratory Fire-Tube Test.

A test in the fire-tube apparatus is conducted substantially as follows: The empty tube is balanced by means of counterweights, *W* and W_1, so that the pointer is in a vertical position and reads 100 on the percentage chart. The specimen, which is first weighed on a separate balance, is then suspended in the tube. This brings the pointer to the left, and the scale is raised or lowered as may be required to bring the end of the pointer on the zero line. The flame is then applied to the specimen with the top of the burner 1 in. below the bottom of the specimen and records are taken, at $\frac{1}{2}$-minute intervals, of the percentage of loss in weight of the specimen and of the temperature at the top of the tube, as shown on the chart and pyrometer, respectively. The flame is removed at the end of 4 minutes, but records are taken from the beginning of the test until two minutes after flaming ceases. Observations are also made of the tendency of any fallen embers to glow.

TYPICAL RESULTS OF TEST

Typical results obtained in the fire-tube test are shown in Fig. 2, where the percentage of loss in weight of the specimen and maximum temperatures developed at the top of the tube during the test are plotted against time of test. An example of the application of the results of the tests to specific problems is shown in Fig. 3, where the absorption of diammonium phosphate in pounds per cubic foot of wood is plotted against the percentage of loss in weight and against maximum temperatures recorded at the top of the tube. If desired, the amount of dry salt may be expressed in percentages of the weight of wood instead of in pounds per cubic foot, but in so doing the amount of moisture in the wood must be taken into consideration in order to afford a true basis for comparison. It is also a simple matter to calculate the actual losses in weight of the specimen in grams or other units from the weight of the specimen, taken just before test, and the percentage losses during test. However, the percentage loss in weight appears to be more significant as a measure of the property of wood to spread flame than the actual loss in weight. Actual loss in weight varies with the density of the wood in contact with the ignition flame, whereas percentage loss shows more clearly the extent of burning beyond the ignition flame.

With the loss in weight and the temperatures at the top of the tube recorded, as the burning proceeds, the rate of burning and increase of temperature can be readily determined (see Fig. 3). The rate of burning may be an even more significant and accurate measure of fire resistance than total loss in weight or maximum temperatures. For example, of two specimens nearly equal in fire resistance, one may burn slowly but eventually be almost completely destroyed, showing a large total loss in weight, and the other may cease to burn after a short time, showing a small loss in weight. If, however, the total loss in weight expressed as a percentage and the maximum temperature are divided by time of occurrence the results are similar. Any one of the three factors: namely, time of ignition and burning, loss in weight, and increase in temperature, may be misleading when taken alone. With the fire-tube apparatus, these three factors can be expressed in definite numerical values and correlated as desired.

DEVELOPMENT AND STANDARDIZATION OF TEST

In the development and standardization of the test, a number of variations of the apparatus and method of procedure have been investigated. The following are the more important factors studied:

Type and size of tube;
Heat source; and
Size and number of specimens burned at one time.

Some data were also assembled on the following factors:

Kind, density, and moisture content of wood; and
Method of sampling material.

Type and Size of Tube:

Several different types of tubes were tried out and a series of tests was made with each. The principal variations in the construction of the tubes investigated were: size; open and closed; insulated and uninsulated; cage and screen bottom; horizontal and vertical slots; and size and number of slots.

Size of Tube.—Tubes of 3, 4 and 5-in. diameters, constructed otherwise alike, were used and the results were compared with results obtained by burning specimens in the open without a tube. Burning in any of the three sizes of tubes was more severe than burning in the open. With the three tubes, the severity of the test varied inversely with the diameter of the tube, the 3-in. tube showing the largest loss in weight and the highest temperatures, and the 5-in. tube the lowest values. It was also found that burning a single specimen in the 3-in. tube gave almost the same results and represented about the same degree of severity as burning five specimens together in the open.

Open and Closed Tubes.—A series of comparative tests was made in tubes with bottom and slots open, bottom and slots closed, and bottom closed and slots open. Closing the tube had the effect of raising the temperature surrounding the specimen. This was shown by temperature measurements with the ignition flame in place but without a specimen. With wood treated with medium to heavy absorptions of fire retardants, the higher temperature caused the flame to spread farther up the specimen and increased combustion. On the other hand, closing the tube reduced the amount of oxygen available for combustion and the temperatures at the top of the closed tube were lower than with the open type of tube. With untreated wood the loss in weight was almost the same for the three types of tubes, which indicated that the temperatures were sufficiently high to drive the gases from the wood, even though complete combustion of the gases did not occur in the tube.

Insulated and Uninsulated Tubes.—A comparison was made in the 3-in. tube with the end and slots open having one tube insulated with about a 1-in. layer of asbestos and the other without insulation. The results of the tests indicated that the conditions in the insulated tube were somewhat more severe than in the uninsulated tube. The insulated tube had the disadvantage, however, in that it changed weight as it was heated, necessitating a frequent rebalancing of the apparatus.

Cage-Bottom and Screen-Bottom Tubes.—Originally a tube was used with a $3\frac{1}{2}$-mesh screen in the lower end of the tube and the burner placed below the screen. It was found, however, that falling embers and charcoal often collected on the screen and interfered more or less with the flame from the burner during test. A modification was made, using a wire cage with solid bottom, as shown in Fig. 1, fastened to the bottom of the tube and the burner inserted through a slot in the side of the cage. It was found that this method eliminated the interference of the embers with the flame and that otherwise there was no pronounced difference in the results from the two tubes.

Position, Size, and Number of Slots.—Slots, 0.10 by $1\frac{1}{2}$ in. in size, were placed horizontally in one tube and vertically in another at intervals of 4.1 in. center to center, on one side of the tube. The test results indicated that the conditions were a little more severe in the horizontal-slot tube than in the vertical-slot tube. In other tubes the size of the slots was progressively increased from the bottom to the top and two rows of slots were used on opposite sides of the tube. The tube with two vertical rows of horizontal slots of uniform size (see Fig. 1) was finally selected as admitting sufficient air and giving most uniform burning.

Heat Source:

Heat may be applied to the specimen in different ways, such as by direct contact with flame, indirectly from an electrical heating element,[1] or by passing heated air into the tube.[2] The flame and electrical heating element have both been tried. The latter was not well suited in the fire-tube apparatus for determining the loss in weight of the specimen at intervals during test and had certain other disadvantages. Furthermore, the flame was considered more nearly comparable to exposure under actual fire conditions.

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[1] H. F. Weiss, "Tests to Determine the Commercial Value of Wood Preservatives," a progress report, reprinted from original communications, Eighth International Congress of Applied Chemistry, Vol. XIII, p. 279 (1912); also R. E. Prince, "Tests on the Inflammability of Untreated Wood and of Wood Treated with Fire-Retarding Compounds," *Proceedings*, National Fire Protection Assn. (1915).

[2] Amédée Lullin, "Recherches sur les Températures d'Inflammation du bois et sur les Enduits Ignifuges," thesis submitted to the Federal Polytechnic Institute, Zurich, Switzerland.

Flames developed from burning a given quantity of inflammable material, such as alcohol, were tried but the height and temperature of such flames were more difficult to control and measure accurately and they were more affected by air currents than flames obtained from gas under pressure. A Bunsen flame using manufactured gas as a fuel was found to be a more reliable heat source.

Control of Heat Source.—It is necessary to control the flame in three respects: (1) temperature, (2) height, and (3) amount of heat evolved. The approximate temperature and height of the flame can be readily measured. Tests were made on untreated wood and wood treated with varying amounts of fire-retardant salts, using yellow and blue flames ranging in height from 6 to 14 in. and in temperatures from 780 to 1085° C. In all cases the top of the burner was placed 1 in. below the bottom of the specimen. The results indicated that the effect of the variation in temperature of the flame within the range used was negligible with untreated or lightly treated wood and small with heavily treated wood. The maximum variation was about 1 per cent loss of weight for 25° C. difference in the temperature of the flame, which occurred with intermediate absorptions of fire retardants. The height of the flame within the range of 6 to 14 in. had no appreciable effect on the results for untreated or lightly treated wood, but for heavily treated wood 1-in. variation in height of flame resulted in maximum variation of 2 to 2.5 per cent in the loss in weight of the specimen, about the same proportion as 1 in. is to the total length of the specimen. In other words, the higher flames caused greater loss in weight because they were in direct contact with a greater proportion of the length of the specimen.

No attempt has been made to measure accurately the amount of heat evolved by the flame. The amount of heat applied is important only in so far as it affects the conditions surrounding the specimen during test. The temperature at the top of the tube with the flame in place but without a specimen has been considered a satisfactory check of this condition. The temperature and volume of flame can be controlled sufficiently by the use of an ordinary burner with the gas orifice and air intake properly adjusted and in this way a relatively uniform temperature at the top of the tube can be obtained.

Temperature calibration curves for the fire tube are shown in Fig.

4. The measurements were made in the center and near the inner side of the tube at various points along the tube with a Bunsen blue flame 11 in. high with continuous inner cone, in place but without a specimen. Under an actual test the lower end of the specimen is subjected to a temperature of approximately 1000° C. (1832° F.) immediately upon applying the flame. Above the flame, the temperature surrounding the specimen gradually decreases to about 180° C. at the top. As the testing proceeds, the temperature at the top of the tube increases more or less depending upon the inflammability of the material, with maximum temperatures of about 800° C.

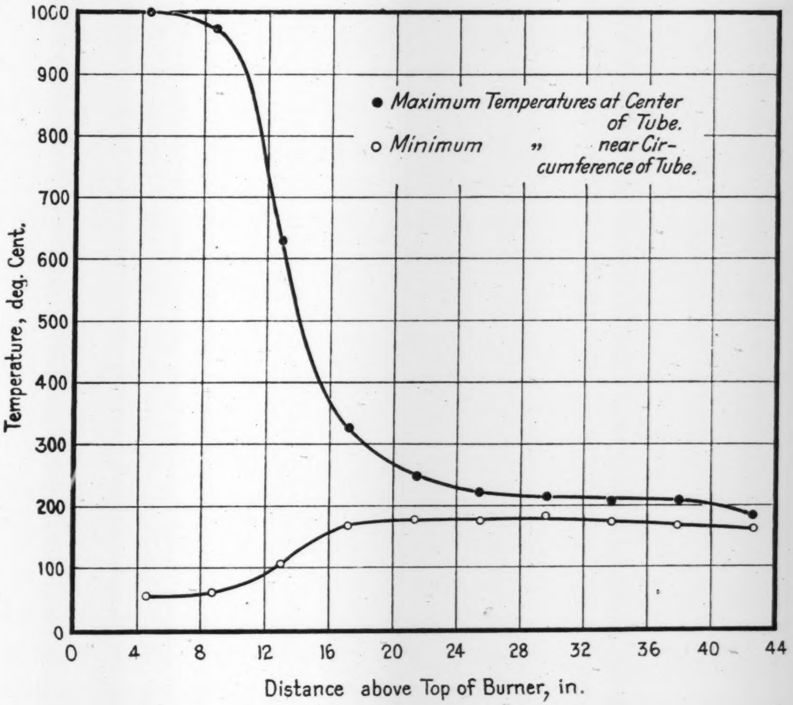

FIG. 4.—Temperature Calibration Curves for Forest Products Laboratory Fire-Tube Apparatus.

Tube.—3-inch diameter, uninsulated, cage bottom type with two rows of horizontal slots, each slot 0.1 by 1½ in. arranged 4 in. apart on opposite sides of the tube.

Heat Source.—A Bunsen blue flame 11 in. in height and 1000° C. at the hottest point. Without specimen in place.

Size and Number of Specimens Burned:

In order to measure the property of wood to carry flame and to support combustion, it seemed advisable to use a comparatively long specimen with the flame applied only to the lower end. Specimens of different lengths and cross-sections have been used. Of three lengths tried, namely, 20, 27½, and 40 in., the longest gave the greatest difference in results between treated and untreated wood and the most accurate measure of differences in effectiveness. The cross-sectional area and form of the specimen were also varied ⅛ to ½ in. in thickness and ½ to 1¼ in. in width. Considered as to the ease of preparation, accuracy, and time required for testing, a specimen ⅜ by ¾ by 40 in. was found to be the most desirable.

Some tests were also made by burning specimens singly, in threes, and in fives. The results showed that burning three or five specimens at a time resulted in a larger loss in weight for the moderate to heavily treated wood than burning a single specimen. When burning five specimens together in the 3-in. tube, it was also noted that a considerable amount of the gas escaped from the tube before it ignited. Several pieces burned together apparently added to each others destruction. This resulted in a continuation of burning beyond the point where a single specimen supplying its own heat ceased to burn. As previously mentioned, however, a single specimen in the tube gave about the same results as burning five together in the open. Furthermore, considering the fact that the specimen is exposed to flame on all sides, it seems that several such specimens, tested together in a tube, would result in a more severe condition than actually prevails in the initiation of fires under most conditions and that burning a single specimen more nearly approximates the severity under actual incipient fire conditions.

Effect of Other Conditions on the Results of Test:

Moisture Content of Wood.—A number of factors, such as moisture content, density, species of wood, and the method of cutting specimens, have an important effect upon the results of the test. Comparative tests made in the fire tube have shown that the moisture content of the wood at the time of the test markedly affects the results. Since the moisture-humidity equilibrium of wood may be influenced by the presence of chemicals, it seemed desirable to condition the wood previous to test to a constant weight under a given atmospheric condition rather than to a fixed moisture content. It is recommended that wood, intended for use in heated buildings, be conditioned at about 80° F. and in a relative humidity of about 30 per cent before test. This brings untreated wood to a moisture content of about 7 per cent, which is close to its average in service in heated buildings.

Heartwood and Sapwood.—Preliminary work has indicated that heartwood and sapwood of some species are sufficiently different in fire resistance to show a different result in the fire-tube test. The presence of heartwood and sapwood also affects the impregnation process and the amount of absorption and distribution of chemicals, the sapwood being the easier to impregnate.

Density.—Density is another factor that is closely related to fire resistance. The rate at which a flame burns a given volume of wood, the percentage of loss in weight within a given time, and the amount of fire retardant required per unit volume of wood, all depend more or less upon the specific gravity of the wood. The heavy, dense woods burn more slowly than woods of low density, other things being equal.

Method of Cutting Specimens.—Sufficient work has not been done to permit of a detailed statement on the method of sampling fire-resistant material. However, in the laboratory experiments with the fire-tube test, a total of 16 specimens, 4 cut from each of 4 boards, has been regarded as sufficient for evaluating a given species and treatment. Where heartwood and sapwood are present, the material should be selected and cut so as to obtain a proper proportion and distribution of heartwood and sapwood in the specimens. Fire-resistant lumber that is to be remanufactured should be surfaced lightly on the edges and sides before cutting test specimens, because of the possibility of uneven absorption and of a greater concentration of fire retardant at or near the surface of the material caused by drying. Surfacing is done in many re-manufacturing processes, so that the test specimens should represent as nearly as possible the wood as it will ultimately ~~be used~~. Tests in the fire tube have shown a distinct difference in effectiveness between specimens cut from the outside and inside of thoroughly impregnated material. The specimens should be cut not less than 1 ft. from the ends of the piece because of the greater concentration of chemical at the ends.

PROPOSED STANDARD METHOD

A choice of the details of the apparatus and method of procedure, such as the size and type of tube, the extent to which it is insulated, the amount of air admitted, the amount and distribution of heat applied to the specimen, and the size and number of specimens tested at one time, depends upon the severity of the test desired. It is desirable, of course, that the conditions of the test be similar to those existing under actual incipient fire conditions, but at the same time they must be such as to differentiate between different fire-resistant treatments. In the absence of data correlating the results of the test more closely with those obtained under actual fire conditions, the following method is proposed:

Tube.—Three-inch diameter, galvanized iron, uninsulated, cage-bottom type with 2 rows of horizontal slots on opposite sides of the tube, each slot 0.10 by 1½ in., arranged 4 in. apart center to center.

Heat Source.—A Bunsen blue flame, from a horizontal, low-form burner, 11 ± ½ in. in height with a tall indistinct inner cone, 1000° C. ± 25° C. at the hottest point and giving a temperature of 180° ± 5° C. at top of tube, top of burner to be placed 1 in. from the bottom of the specimen.

Specimen.—Single, ⅜ by ¾ by 40 in. Specimen to be conditioned to constant weight under uniform humidity and temperature conditions to bring it to about 7 per cent moisture content and accurately weighed immediately before test.

Duration of Test and Measurement of Result.—The ignition flame to remain under specimen 4 minutes; percentages of loss of weight and temperatures to be taken at ½-minute intervals from beginning of test until 2 minutes after flaming ceases; loss in weight to be noted at time flaming stops; temperatures to be taken by placing a thermocouple not more than 1 in. above the center of the top of the tube at the beginning of the test.

COMPARISON OF THE FIRE TUBE WITH OTHER FORMS OF TEST

Comparative tests have been made between the fire tube and other forms of test. An apparatus used by Prince[1] was tried. The method consists in exposing a specimen of wood in the center of an insulated quartz tube, which is electrically heated, and the time required for the flame to develop and the time of flaming after the specimen is removed from the heated tube is noted. The results did not show a definite relation between the time required for the flame to develop and the fire resistance of the material as measured by the fire-tube test. Neither did the time of burning after removal from the ignition chamber appear to offer a consistent measure of the relative effectiveness of fire-retardant treatments, although the duration of blazing became less, in general, as the amount of chemical increased.

It seemed desirable to make a comparison between the fire-tube test and the shavings, crib, and timber tests which are in use as acceptance tests by the Borough of Manhattan, New York City. A series of tests was made using southern yellow pine untreated and treated with amounts of zinc chloride ranging from about 1 to 5¼ lb. of salt per cubic foot of wood. The southern yellow pine, which was all sapwood, was thoroughly impregnated by a pressure treating process. Specimens for each of the four tests were then cut from the same boards. Four boards were tested for each treatment. The shavings, crib, and timber tests were made in accordance with the procedure

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[1] R. E. Prince, "Tests on the Inflammability of Untreated Wood and of Wood Treated with Fire-Retarding Compounds," *Proceedings*, National Fire Protection Assn. (1915).

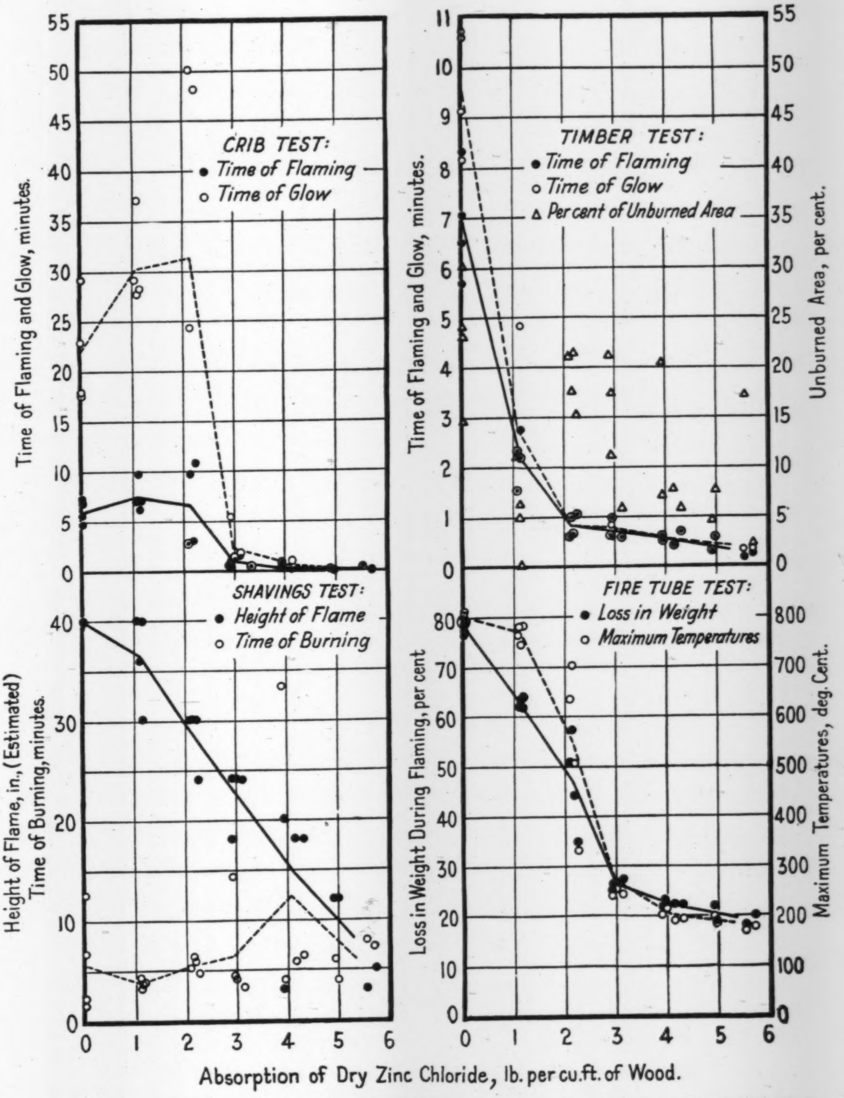

FIG. 5.—Results Obtained with the New York Tests and with the Fire-Tube Tests.

Tests made on southern yellow pine sapwood treated with zinc chloride. Absorptions expressed as pounds of dry salt per cubic foot of wood.

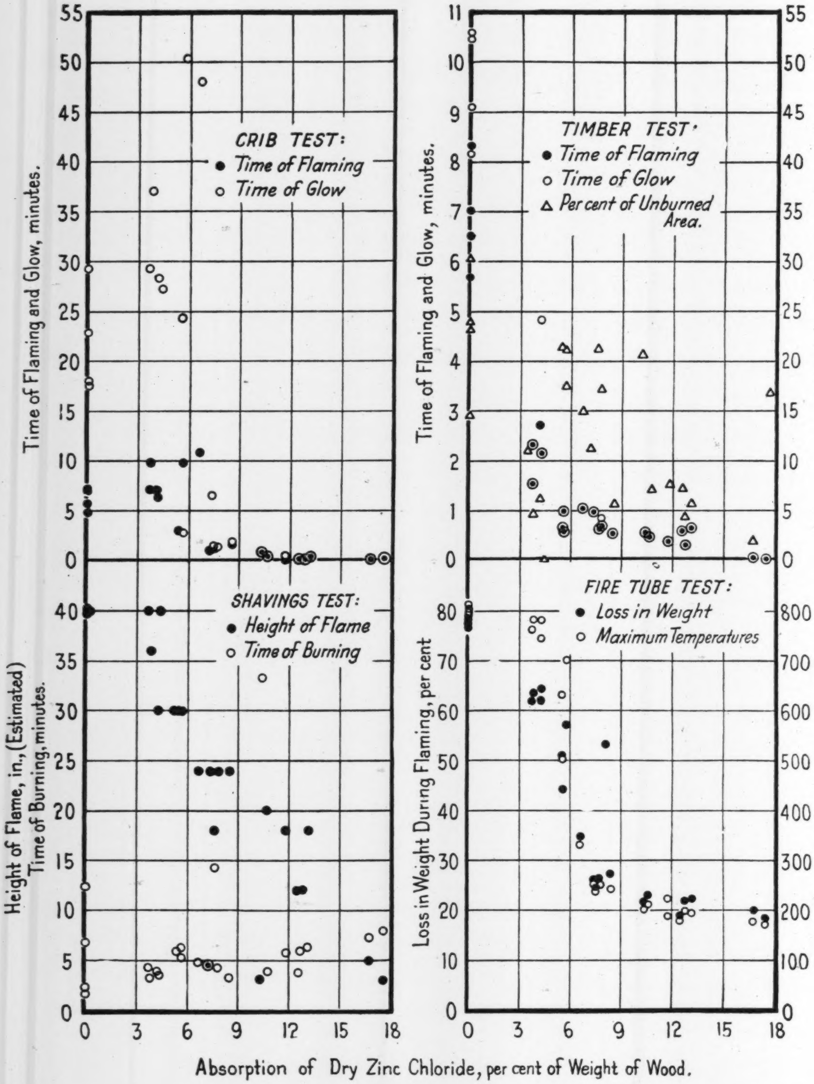

FIG. 6.—Results Obtained with the New York Tests and with the Fire-Tube Tests.

Tests made on southern yellow pine sapwood treated with zinc chloride. Absorptions of dry salt expressed as percentage of the original weight of the wood.

adopted by the Board of Standards and Appeals of New York City.[1] The results obtained are shown in Figs. 5 and 6. In Fig. 5, the units employed as a measure of effectiveness in the various tests are plotted against absorption of dry salt in pounds per cubic foot of wood. In Fig. 6, the amount of dry salt is expressed as percentages of the weight of the wood.

Crib Test:

In the crib test the results are measured in terms of time of flame and glow persistency of the specimen after the ignition flame is removed. These measures showed no consistent difference in fire resistance of wood untreated and treated with light absorptions of salt, a marked difference between 2 and 3 lb. of salt per cubic foot (about 6 to 9 per cent of the weight of the wood) and small differences for all absorptions above 3 lb. per cu. ft.

Timber Test:

The results of this test are judged by the cross-sectional area of wood unburned at the most burned section of the specimen and by the persistence of flame and glow after the ignition flame is removed. None of the material tested passed the requirement as to the amount of unburned area. The percentage of unburned area was quite inconsistent in reference to treatment and was apparently affected as much or more by the density of the wood and other factors as by the amount of fire retardant in the wood. The average persistence of flame and glow showed a rather definite relation to the amount of fire retardant in the wood, but again there was a large variation among individual boards in the untreated or lightly treated material.

Shavings Test:

In the shavings test, the results are judged by the maximum height of flame and by the time that combustion continues in the shavings. Considerable care was used to obtain shavings of uniform thickness and form, which is very difficult to do by hand, and to conduct the test in a standard way. In spite of this, the time of burning showed no apparent relation to the amount of fire retardant present. The height of the flame could not be measured accurately and had to be estimated. The estimated values, although dependent upon the judgment of the operator, did show a fairly definite relation to the amount of fire retardant present. However, in another series of tests in which the thickness and form of shavings was purposely varied there was a marked difference in results obtained from boards receiving the same treatment. There are so many uncontrolled factors in this test, such as the fineness and form of shavings, quantity of shavings, distribution of shavings in the container, and degree of packing and measurement of height of flame, as to make the test of doubtful value for differentiating between fire-resistant treatments.

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[1] *Bulletin*, Board of Standards and Appeals of New York City, Vol. XIII, No. 1., January 3, 1928, and amendment thereto adopted in March, 1928.

Fire-Tube Test:

The results obtained in this series of tests in the fire tube are fairly typical. Individual boards treated with about the same amount of fire retardant showed about the same percentage loss in weight and the same maximum temperatures, except in the region of the 2-lb. absorption of dry salt per cubic foot of wood (about 6 per cent of wood by weight). (See Figs. 5 and 6.) The fire resistance of the southern yellow pine containing 2 lb. of zinc chloride was just sufficient that the specimen might or might not support combustion, depending upon the density of the wood and perhaps other factors. Consequently there was greater variation among individual boards at this absorption. When the amount of fire retardant was expressed as a percentage of the weight of the untreated air-dry wood (see Fig. 6), the test results were more consistent. The wood that gave the test results shown in Figs. 5 and 6, had been conditioned to a fairly uniform moisture content before treatment. If there had been wide variations in the moisture content of individual boards before treatment, as may be the case in commercial lots of lumber, the absorptions by percentage of weight of air-dry wood would have been less significant and less consistent.

The fire-tube test, in comparison with the crib, timber, and shavings tests, gave more consistent results both as regards boards of the same treatments and boards treated with different amounts of the fire retardant. Moreover, the results can be more accurately determined and the test can be made much more quickly. The persistence of flame and glow, shown by the crib and timber tests, are apparently measures of the same essential property of flammability as is measured more accurately in the fire-tube test. It is possible that the timber test, or a similar test, might give some information about the properties of fire-retardant wood not revealed by the fire-tube test, but it is apparent that density of wood and possibly other factors must be carefully controlled to obtain comparative results. As a measure of the property of wood to support combustion, that is, to cause the spread of flame and to contribute to an increase in temperature under fire conditions, the Forest Products Laboratory fire-tube test is reliable, consistent, and quickly and easily made.

DISCUSSION

MR. C. H. HONECK[1].—In the comparison between the tube test and the one in vogue in New York City, it strikes me that the timber test in New York City gives a better idea of the strength of the timber after it has stood the flame a certain time than the tube test would. I do not know whether that can be verified or not, but under the New York test the unburned area is supposed to be 45 per cent in soft wood and 55 per cent in hard wood, and we have in our tests up to 90 per cent by actual tests. I should like to ask the authors whether in their tube test that can be determined as well as under the present timber test?

MR. T. R. TRUAX.[2]—In answer to that question, we do not propose the fire-tube test as a measure of the strength of wood under high temperatures or under conditions existing under fire hazards. I do not think the fire-tube test measures the reduction in strength due to high temperatures nor the time that a timber will stand up in a fire. However, we feel that the tube test does measure the properties of materials that spread flame and that contribute to an increase in temperatures, which I conceive to be rather important from a fire-resistance standpoint. There are other properties for which other tests may have to be devised, or continue to use tests now in existence.

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[1] President, The Batavia and New York Wood Working Co., Batavia, N. Y.

[2] Senior Wood Technologist, Forest Products Laboratory, Madison, Wis.

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